

APPENDIX B

GEOTECHNICAL REPORT





COSTA MESA ▪ SAN DIEGO ▪ TEMECULA ▪ LOS ANGELES

March 29, 2002
J.N. 147-95

MR. AND MRS. RICK LAWRENCE
19191 Lawrence Canyon
Star Route Box 1038
Orange, CA 92661

Attention: Mr. Jim Walton

Subject: Update of Geologic Reconnaissance and Environmental Impact Assessment Report of Lawrence Property, 19191 Lawrence Canyon Road, County of Orange, California.

Reference: Preliminary Geologic Reconnaissance and Environmental Impact Assessment of Lawrence Property, Santiago Canyon Road, Orange County, California; report by Leighton and Associates, Inc., dated February 5, 1983.

Dear Mr. and Mrs. Lawrence:

At the request of Mr. Neno Grguric with Andrade Architects, this letter provides an update of the preliminary geologic reconnaissance and environmental impact assessment report for the subject property prepared by Leighton and Associates in February of 1983 (Reference).

In order to update this previous report, we have reviewed the previous report in detail and have performed several recent reconnaissances of the site in order to evaluate existing conditions. It should be noted that since the preparation of this previous report, some improvements have been made within the site. The owners of the property have entered into an agreement to plan check and permit all existing grading and structures that require permits. Consequently, numerous geotechnical investigation reports have been prepared by Action Geotechnical, Strata-Tech and our own firm to

evaluate existing geotechnical conditions within the site and to begin the process of obtaining permits for the grading and construction that has occurred (see Literature Reviewed).

Based on our review, the referenced report by Leighton and Associates described the site as consisting of Aliso Creek within the eastern one-third of the site and a moderately steep to steep ascending natural slope within the western two-thirds of the site. At the time of Leighton's report, the site was occupied by two dwellings and several paved and dirt access roads. The more gently sloping lower portions of the property had been planted with fruit or nut trees while the moderately steep to steep natural slope areas were covered by native weeds, grasses, shrubs and trees.

The report indicated that the lower portions of the site along Aliso Creek are underlain by alluvium and slopewash materials with local deposits of fill along the access roads while the natural slope is underlain by sedimentary bedrock materials of the Sespe and Vaqueros Formations which are mantled by a thin layer of topsoil or slopewash. The bedrock materials were described as consisting of massive sandstones with thickly bedded siltstone and clay-rich interbeds. The bedrock was described by Leighton as striking northwesterly and dipping towards the southwest at angles that range predominantly from 25 to 45 degrees and occasionally as steep as 70 degrees.

The Leighton report indicated that no major or active faults are known or suspected to cross the subject site and that the Whittier-Elsinore and Newport-Inglewood Faults would likely produce the most significant ground motions within the site during a seismic event.



Leighton concluded that the hazard from ground rupture due to fault displacement is considered negligible due to the distance of the site from nearby active faults and that the effects of anticipated ground shaking within the site can be satisfactorily mitigated through building design that conforms to the Uniform Building Codes. Leighton also concluded that the potential for any secondary seismic hazards such as liquefaction, flow-sliding, seismic-induced settlement, and ground lurching or cracking are also considered negligible.

Leighton also concluded that the site is considered to be grossly stable due to the favorable geologic structure; however, oversteepened slope areas underlain by surficial soils and weathered bedrock may be subject to shallow failures. Leighton additionally noted that the lower portion of the property along Aliso Creek is located within a 100-year flood zone and all habitable structures should be located outside of the flood zone.

Based on our knowledge of the surface and subsurface geologic conditions within the site as obtained during our previous geotechnical investigations within the site (see Literature Reviewed), we concur with the findings and conclusions of the referenced Leighton report. Our recent site visits indicate that conditions within the site are essentially the same as those described by Leighton and Associates with the exception of the construction of numerous new buildings and other structures within the site as mentioned previously. Since conditions within the site are essentially the same, the previous findings and conclusions by Leighton and Associations are still considered to be applicable to the subject site.



MR. AND MRS. RICK LAWRENCE

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It should be noted that specific geotechnical reports will be prepared for the various portions of the site where grading or construction has been performed. These reports will be submitted to the Grading Section of the County of Orange Planning and Development Services Department for their review and approval.

We hope this letter serves your needs at this time. Should you have any questions, please call our office.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.


David Hansen
Senior Project Engineer
RCE 56591



DH/we

Distribution: (1) Addressee
(4) Andrade Architects

cc: 2002\100\147-95B.LTR



LITERATURE REVIEWED

- 1) Geotechnical Investigation of Parking and Berm Areas, 19191 Lawrence Canyon Rd., by Action Geotechnical, dated November 14, 1989, W.O. 557701.
- 2) Response to County Review Sheet dated December 14, 1989, by Action Geotechnical, dated January 11, 1990, W.O. 557701-A.
- 3) Change in Geotechnical Consultants, 19191 Lawrence Canyon Rd., by Strata-Tech, Inc., dated August 7, 1990.
- 4) Geotechnical Investigation of Phase I, Rancho Los Lomas Property, 19191 Lawrence Canyon Rd., County of Orange, by Strata-Tech, Inc., dated August 1, 1990, W.O. 27190.
- 5) Geotechnical Investigation, Proposed Residential Sites, 19191 Lawrence Canyon Rd., County of Orange, by Strata-Tech, Inc., dated September 18, 1990, W.O. 30390.
- 6) Response to County of Orange Review Sheet dated September 20, 1990, for 19191 Lawrence Canyon Rd., OCPC# 648-89G, by Strata-Tech, Inc., dated March 26, 1991, W.O. 27190-01.
- 7) Response to County of Orange Review Sheet dated May 8, 1991, Residential Sites, GPC 640-89G, by Strata-Tech, Inc., dated October 4, 1991, W.O. 30390-A.
- 8) Response to County of Orange Review Sheet dated April 19, 1991, Phase I Investigation, GPC# 640-89G, by Strata-Tech, Inc., dated October 8, 1991, W.O. 27190-02.
- 9) Digest of Geotechnical Conditions and Recommendations, 19191 Lawrence Canyon Rd., by Strata-Tech, Inc., dated January 15, 1992, W.O. 27190-03.
- 10) Geotechnical Investigation of Phase II, Rancho Los Lomas, 19191 Lawrence Canyon Rd., County of Orange, by Strata-Tech, Inc., dated January 21, 1992, W.O. 27190-04.
- 11) Response to County of Orange Review Sheet dated February 27, 1992, by Strata-Tech, Inc., dated April 22, 1992, W.O. 27190.
- 12) Response to County of Orange Review Sheet dated May 8, 1992, by Strata-Tech, Inc., dated July 15, 1992, W.O. 27190-05.
- 13) Variance Package, Oversteepened Slopes, 19191 Lawrence Canyon Rd., by Strata-Tech, Inc., dated July 22, 1992.
- 14) Geotechnical Data for Variance Request, Parking Area, by Strata-Tech, Inc., dated August 5, 1992, W.O. 27190-06-1.
- 15) Geotechnical Data for Variance Request, Berm Area, by Strata-Tech, Inc., dated August 5, 1992, W.O. 27190-06-2.

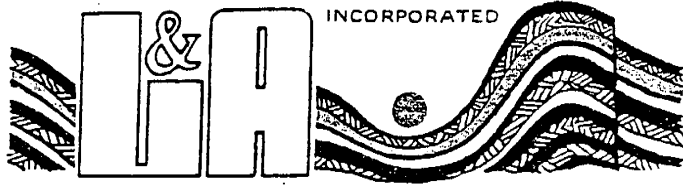
LITERATURE REVIEWED

- 16) Geotechnical Data for Variance Request, Dam Areas, by Strata-Tech, Inc., dated August 5, 1992, W.O. 27190-06-3.
- 17) Response to County of Orange Review Sheet dated August 27, 1992, for Variance of Oversteepened Slope (S1-S18) by Strata-Tech, Inc., dated September 7, 1992, W.O. 27190-07.
- 18) Response to County of Orange Review Sheet dated August 12, 1992, by Strata-Tech, Inc., dated September 7, 1992, W.O. 27190-08.
- 19) Geotechnical Data for Setback Variance Request for Buildings A, B, D, E, G, R, U, X and Y, 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated September 23, 1992, W.O. 27190-09.
- 20) Geotechnical Investigation of Existing Pool and Descending Slope, 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated October 26, 1992, W.O. 27190-10.
- 21) Limited Investigation of the Hardscape at 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated October 28, 1992, W.O. 27190-11.
- 22) Response to Geotechnical Review Sheet, dated September 30, 1992 for 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated October 28, 1992, W.O. 27190-12.
- 23) Geotechnical Investigation of Buildings H, A-A, and A-B (Phase III), Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated February 27, 1995.
- 24) Structural Pavement Sections, Fire Access Roads, Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated August 24, 1995.
- 25) Geotechnical Investigation, Proposed Greenhouse, Tool Storage Room, Restrooms, and Existing Building "C", Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated October 20, 1995.
- 26) Geotechnical Investigation, Proposed Building A-D, Covered Bridge, Entrance Gate, and Existing Building "E", Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated November 9, 1995.
- 27) Geotechnical Investigation for Design and Construction of Proposed Bridge Culverts and Evaluation of Existing Access Roads and Creek Banks, Rancho Las Lomas, 19191 Lawrence Canyon, Silverado Area of the County of Orange, California; report by Petra Geotechnical, Inc., dated October 17, 2000.
- 28) Geotechnical Investigation for Design and Construction of Proposed Deck and Trellis, Rancho Las Lomas, 19191 Lawrence Canyon, Silverado Area of the County of Orange, California; report by Petra Geotechnical, Inc., dated October 19, 2000.

LITERATURE REVIEWED

- 29) Updated Geotechnical Recommendations, Proposed Entrance Structure, Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; letter by Petra Geotechnical, Inc., dated March 26, 2001.
- 30) Geotechnical Investigation, Proposed Conservatory, Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated October 29, 2001.

LEIGHTON and ASSOCIATES



SOIL ENGINEERING

TESTING

GEOLOGY

ENVIRONMENTAL SCIENCES

PRELIMINARY GEOLOGIC RECONNAISSANCE AND
ENVIRONMENTAL IMPACT ASSESSMENT OF
LAWRENCE PROPERTY,
SANTIAGO CANYON ROAD,
ORANGE COUNTY, CALIFORNIA

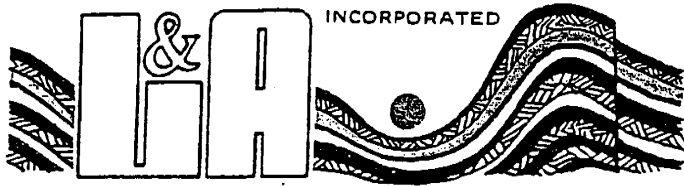
February 4, 1983

Project No. 1820248-01

Prepared for:

MR. R. L. LAWRENCE
Star Route, Box 1038
Orange, California 92667

LEIGHTON and ASSOCIATES



SOIL ENGINEERING

TESTING

GEOLOGY

ENVIRONMENTAL SCIENCES

February 4, 1983

Project No. 1820248-01

TO: Mr. R. L. Lawrence
Star Route, Box 1038
Orange, California 92667

SUBJECT: Preliminary Geologic Reconnaissance and Environmental Impact Assessment of Lawrence Property, Santiago Canyon Road, Orange County, California

In accordance with your request and authorization, we have completed a preliminary geotechnical assessment study of the subject property, as outlined in our proposal of April 23, 1982. Submitted for your review and use in the preparation of an environmental impact report (as well as for general land-use planning guidance) are six copies of our report which documents the research, analysis of field reconnaissance, and evaluation of the potential geologically related environmental impacts, hazards and constraints to the proposed land uses, or possible improvements related to them. This report, which was prepared in accordance with the CEQA and Orange County guidelines for EIRs, summarizes the findings of our analyses and presents possible mitigation measures to minimize the potentially adverse impacts identified.

We appreciate the opportunity to be of service to you. Should you have any questions regarding this report or require further information, please do not hesitate to contact the undersigned.

Respectfully submitted,

Richard Lung, Vice President
Principal Engineering Geologist, EG III

Reviewed by: Bruce R. Clark
Principal Engineering Geologist, EG 1073

RL/BC/sdb

Attachments: Figures 1, 2 and 3
Tables 1 and 2
Appendix A

Distribution: (6) Addressee

1.0 SUMMARY OF FINDINGS AND CONCLUSIONS

- Our geotechnical analysis of the site and assessment of potential environmental impacts posed by the proposed future commercial, and existing agricultural land uses, have revealed no geologic, soil or hydrogeologic constraints, hazards or problems sufficiently serious to preclude the proposed uses or cause unavoidable adverse environmental impacts, provided that currently applicable codes are observed and appropriate construction methods are utilized during development or improvement of the site.
- Although there are no existing landslides or major slope stability problems anticipated to affect the property, surficial slope failures (e.g., mudflows or accelerated erosion) could affect the steeper portions of the site, particularly if runoff is not appropriately controlled. Site-specific geotechnical investigations will be necessary to evaluate and review proposed development plans, and to recommend mitigation measures, if necessary.
- Additional on-site sewage disposal systems required by future dwellings or other facilities are considered feasible on a limited or interim basis, but should be connected to a sanitary sewer when it becomes available.
- No special fault or seismic shaking hazards are anticipated to affect the site or require special building restrictions or building designs.



2.0 INTRODUCTION

2.1 Objective and Scope

The purpose of our study was to provide a preliminary geotechnical assessment of potential environmental impacts of the proposed commercial and residential/agricultural land uses of the subject property (known as Rancho Las Lomas), and to provide necessary documentation for a zone change application. This report identifies potential geologic, seismic, soil and hydrologic hazards and constraints; evaluates potential impacts on mineral resources; and presents possible mitigation measures, where necessary. The scope of our studies included the following:

1. Review of pertinent published maps and reports, including the Orange County Seismic Safety Element and the Foothill/Trabuco Plan EIR; refer to Appendix A for a complete list of references.
2. Analysis of sequential stereoscopic aerial photographs to document slope and site history.
3. Geological site reconnaissance and field mapping.
4. Data analysis, impact assessment and report preparation.

2.2 Proposed Land Use

It is our understanding that the existing agricultural zoning of the property is proposed to be changed, in part, to a commercial zoning to accommodate the private zoo usage contemplated. The commercial zoning would be applied to the smaller portion of the property (shown as the 4⁺-acre part, designated Parcel I on Figure 2); and the residual parcel would remain agriculturally zoned, with the possibility that residential structures might be proposed on it in the future.

2.3 Site Description

The subject property, comprised of approximately 21.4 acres, is located along the west side of Santiago Canyon Road (formerly Modjeska Road), about 1,000 feet north of its intersection with Live Oak Canyon Road (Cook's Corner); refer to the Index Map, Figure 1, which depicts the site location and the general topography of the area. Details of the site topography are illustrated on the base map of the Geologic Map (Figure 2).

The natural terrain of the site is characterized by gentle to moderately sloping hillsides adjoining the canyon bottom of Aliso Creek in the eastern one-third, and steeper, more rugged hillside ascending westward in the remaining two-thirds of the site. The maximum topographic relief (elevation difference between the highest and lowest points) within the property is approximately 285 feet. Slope gradients range from about 20 percent (5 horizontal to 1 vertical ratio) near the canyon bottom and on the gentle knoll in the north portion of the property, to about 50 percent (2 horizontal to 1 vertical ratio) and locally steeper on the ridge flanks.



1820248-01

INDEX MAP

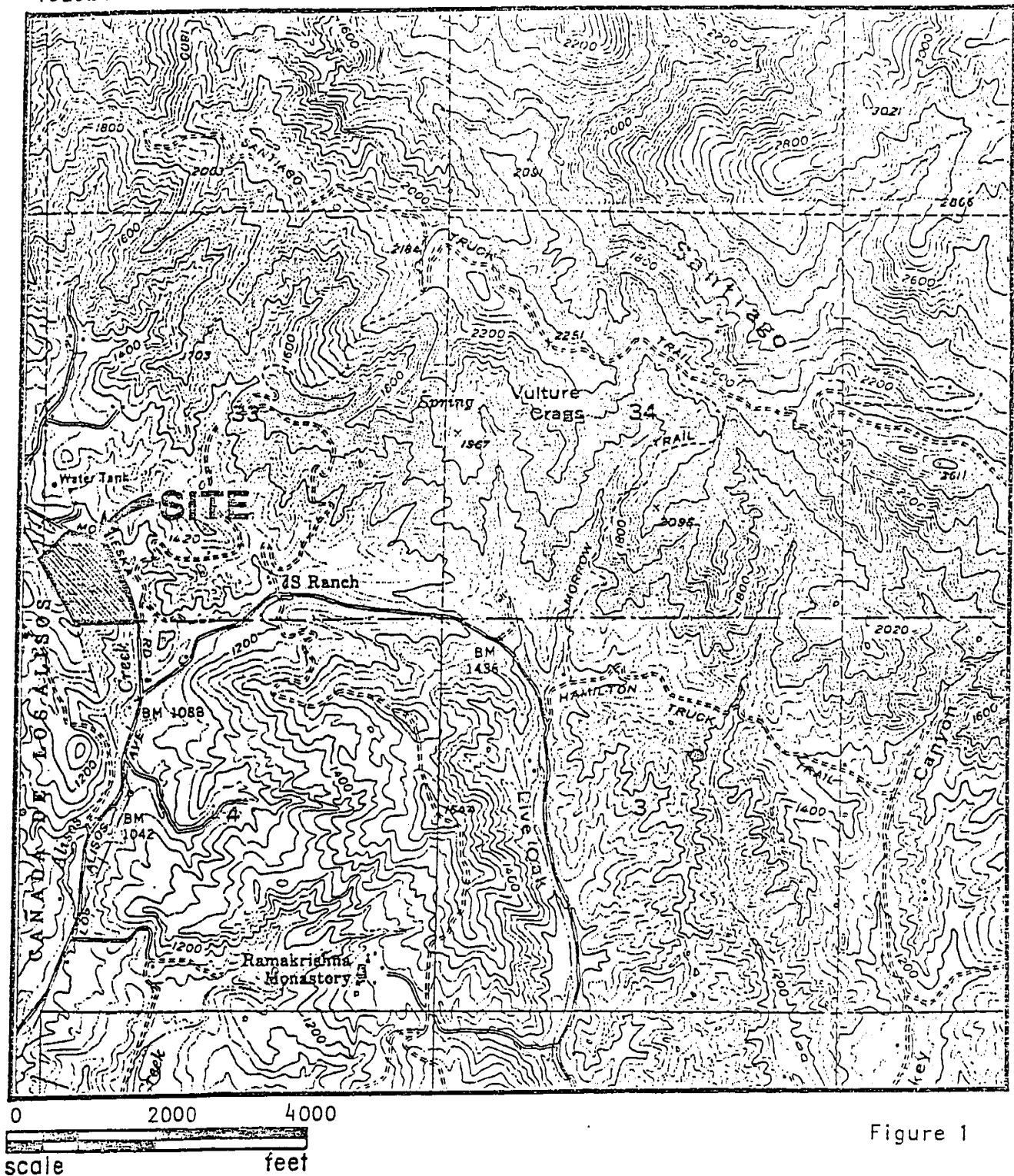


Figure 1

INDEX MAP
OF
LAWRENCE PROPERTY
SANTIAGO CANYON ROAD
ORANGE COUNTY, CALIFORNIA

BASE MAP: U.S.G.S. Santiago Peak Quadrangle



Native vegetation covering most slopes includes annual grasses, scrub oak and chaparral, with some large trees on the steeper flanks of the ridge in the central and south portions of the site, and along the canyon bottom. Water-seeking plants (phreatophytes) are common near the creek, where shallow groundwater tends to be present following seasonal rains. Citrus and various other types of fruit or nut trees are planted in the more gently sloping portions of the property.

Man-made features on the site include two dwellings which are presently occupied and a network of paved and dirt access roads in good repair throughout the site. Portions of the site are presently being used for agricultural purposes and are accompanied by irrigation lines, small drainage dikes and a small water storage tank. Fence lines skirt the perimeter of the property and portions of the area surrounding the existing dwellings.

2.4 Historic Activities

In the past, the subject site has been used for cattle grazing. Agricultural land use was probably initiated about the time the first dwelling was constructed, which is estimated to be in the 1950s. Comparison of old (1939) and new (1983) aerial photographs indicates that the Santiago Canyon Road has been realigned and apparently widened slightly, probably sometime after 1974. Recently, there has been a considerable amount of grading just southwest of the site (higher on the ridge), which is reportedly for a housing development.



3.0 GEOLOGIC SETTING

3.1 Regional Geology

The study area is situated on the southern flank of the Santa Ana Mountains, in the northwest Peninsular Range Province of southern California. This region is composed of a sequence of marine to nonmarine sedimentary strata, ranging in age from late Cretaceous to early Miocene, which were uplifted and tilted southwestward at moderate to steep angles.

Significant faults in the region include the Aliso fault (two miles east of the site), and a zone of unnamed faults along the projected trace of the Cristianitos fault, passing west of the site within 1.0 mile. More distant major potentially active and active faults include the Whittier-Elsinore, Newport-Inglewood, San Jacinto, Sierra Madre and San Andreas faults. No significant faults are known to transect the subject properties.

3.2 Bedrock Formations

Bedrock at the site consists of a transitional, interfingering portion of the Vaqueros and Sespe Formations designated "Tvs" on the accompanying Geologic Map. The contact mapped by others (see Reference 8, Appendix A) was not apparent by surface mapping of the site and exposures in the westerly portion of the site are characteristic of Sespe Formation in the area, rather than the Vaqueros Formation, as mapped by others. The bedrock is comprised of light-colored, massive sandstone with thick-bedded siltstone and local clay-rich interbeds. Although the two formations are of similar composition, the older Sespe Formation is of continental origin and lacks marine (mollusk) fossils, which are sometimes found in the Vaqueros Formation.

3.3 Soil Types

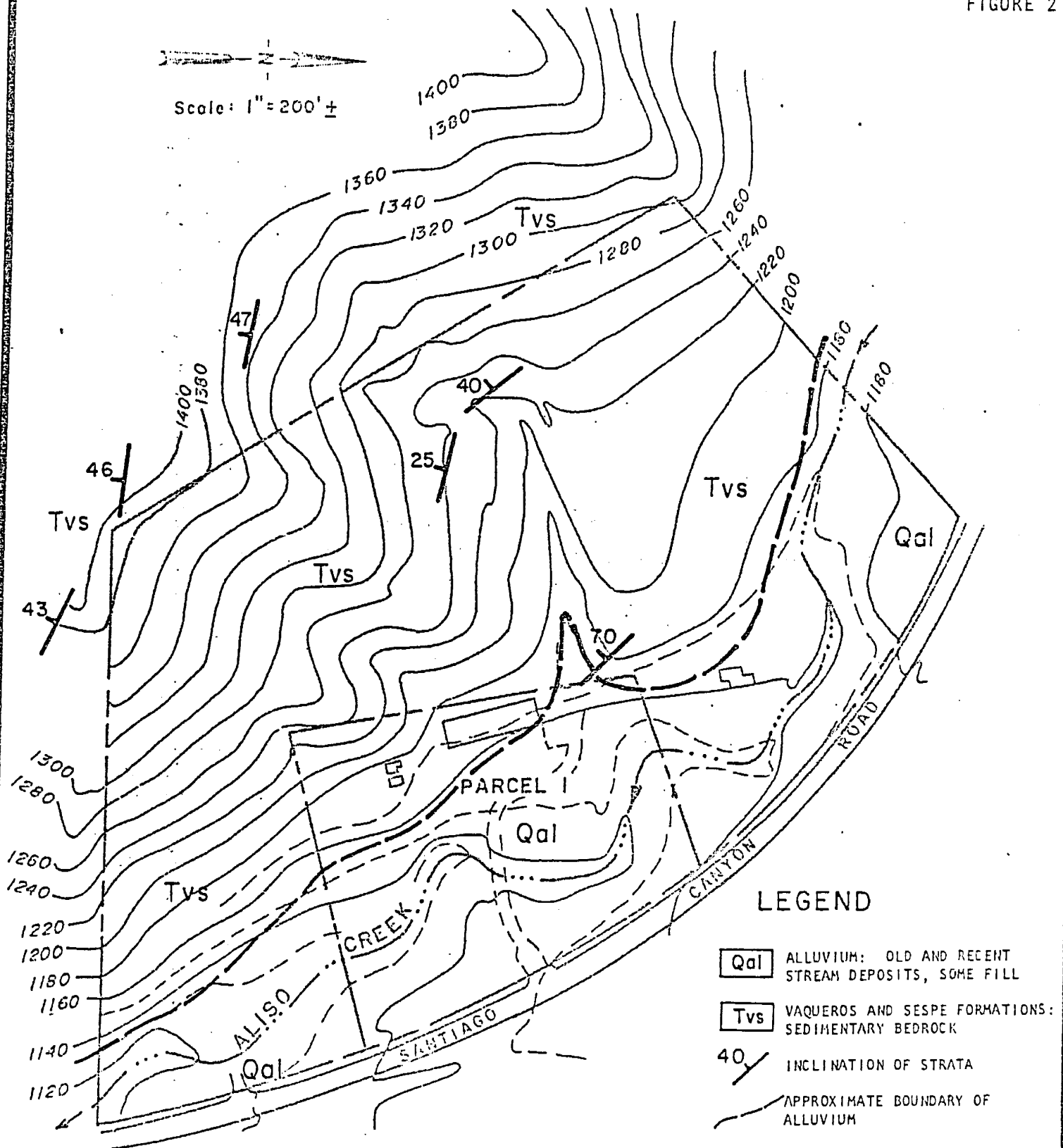
Alluvium and slope wash deposits (Qal on Figure 2) occur in the valley bottom areas in the central part of the site. Ephemeral tributaries contain little or no alluvium. These deposits are of varying composition, generally unconsolidated, and of variable thickness greater than 4 feet. Included in this mapped unit are minor amounts of younger alluvium in the active stream channels and local occurrences of older alluvium along the flanks of Aliso Creek and below Santiago Canyon Road. Some fill is present along access roads and is probably associated with the planted grove areas and the road realignment grading.

The residual surface soils or topsoils are generally less than 2 to 3 feet thick. In terms of agricultural classification, two main types of surface soils have been mapped by the Soil Conservation (Reference 22). The most predominant types at the site are the Alo clay, which mantles nearly all of the steeper slopes, and the Sorrento loam, developed on the alluvial and slope wash deposits lining the canyon bottom areas. The clay typically has a low permeability and a high shrink-swell potential. Such characteristics are not particularly desirable from an engineering standpoint and can present some building site development limitations (Reference 22). The loam generally has less development constraints and is more



GEOLOGIC MAP
LAWRENCE PROPERTY
SANTIAGO CANYON ROAD, ORANGE COUNTY

FIGURE 2



NOTE: BASE MAP MODIFIED FROM PARCEL MAP OF 10-9-73 AND TOPOGRAPHIC MAP SUPPLIED BY OWNER

suited for agricultural purposes than the clay. Both, however, are rated as having a high erosion potential where the soil is bare.

3.4 Geologic Structure

The bedrock formations underlying the subject property represent a relatively simple geologic structure, forming a consistent homoclinal sequence of strata inclined southwestward. The bedding trends (strikes) generally northwest and inclines (dips) southwestward between about 25 and 45 degrees, but locally as steep as 70 degrees. Local steepening of beds probably results from broad warps in the otherwise homoclinal structure, as is characteristic of the general area. Measurements of the bedding inclination within and upslope of the property are shown on Figure 2.

3.5 Faults

No major or active faults are known or suspected to cross the subject property, based on our field reconnaissance and referenced maps or reports. The 4-S Ranch fault, a major fault once believed to exist just northeast of the site (References 9 and 16) has been reinterpreted as a normal formation boundary (R. Miller, California Division of Mines and Geology, personal communication).

Although fractured bedrock material is exposed in the upper 5 to 10 feet of canyon walls in the central portion of the site, it appears to result from weathering and probable slope creep, rather than from faulting.

The nearest major faults are approximately 800, 4,500 and 5,500 feet west of the site. The latter two are probable extensions of the Cristianitos fault zone and may be considered potentially active, based on data further south, along the zone (refer to Figure 3). In the absence of known active faults in the vicinity, however, there have been no Alquist-Priolo Special Studies Zones established on or near the site. Therefore, no mandatory requirement for the special investigation of fault rupture hazards imposed by such zones applies to the subject property.

3.6 Seismicity

Past earthquake activity, as documented by maps plotting earthquake epicenters, their associated magnitudes, and other seismic data, serves as a guide to probable future seismic activity at a given location, provided that the length of record is long enough to be representative. In some cases, particularly for assessing the seismic risk associated with potentially active faults (for which the geologically recent seismic record is not typical of past activity), another method of analysis is required.

The major active faults in southern California which are considered likely sources of future seismic activity that might produce significant ground shaking at the site are the Whittier-Elsinore, Newport-Inglewood, San Andreas, San Jacinto and Sierra-Madre. Refer to Figure 3 for the location of these faults and epicenters of significant earthquakes relative to the site. A review of earthquake epicenters by Morton and others (Reference 10) indicates a series of moderate earthquakes with magnitudes of 4.0 and 5.5 occurred just northeast of the site. This cluster of epicenters has been attributed to seismic activity on the Whittier-Elsinore fault in



1938 and most likely resulted in significant ground shaking at the site. The map also indicates a small 2.0 magnitude event just southwest of the site along the Cristianitos fault zone; however, this has not been generally accepted as proof that the fault is active.

Table I summarizes the seismic parameters of each of the major active faults listed above. In the event that the largest probable earthquake occurs on any of these faults, the seismic shaking to be felt at the site will depend upon the magnitude of the earthquake, the distance to the epicenter, and the site response characteristics. The key seismic parameters which influence the site response and the design of structures to resist ground motion are acceleration, predominate period, and duration of strong shaking.

Additional information shown on Table I includes historic earthquakes and estimated recurrence intervals for maximum credible earthquakes (larger than maximum probable earthquakes which generally apply only when considering the design of critical structures, such as dams, nuclear reactors, or hospitals). Private residences, commercial buildings, schools and nearly all other types of high-occupancy structures are designed for the maximum probable event, or in accordance with current building code requirements.

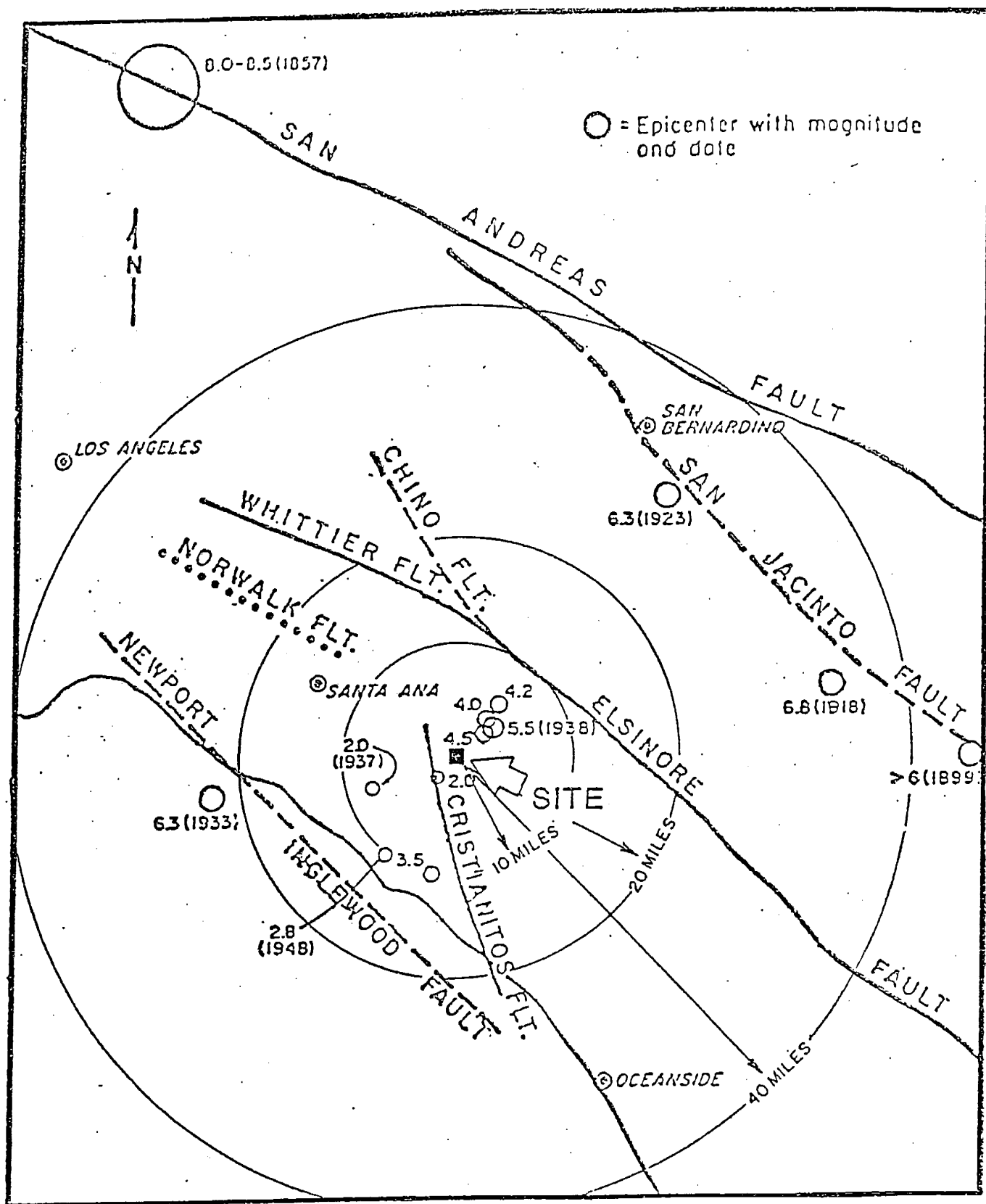
3.7 Slope Stability

Existing slope stability conditions within the site are generally favorable, as evidenced by the absence of significant landslides. The on-site bedrock formation, however, is susceptible to erosion and relatively shallow, slump-type failures which typically affect the near-surface soils or weathered bedrock materials, particularly where they underlie the steeper slopes in the westerly portion of the site (refer to Figure 2). Erosion can occur along existing drainage channels and unpaved access roads, particularly from heavy runoff during the rainy season.

An earlier published map (Reference 9) indicated the possible existence of a large landslide underlying the more gently sloping knoll in the north portion of the site. Subsequent mapping by the state (Reference 8) and by us during our recent reconnaissance revealed no supportive evidence that a slide is present.

The composition and geologic structure of the underlying bedrock formations are generally not conducive to major landslides, provided that zones of weakness (stratified clay and siltstone) and potentially adverse slope angles or directions are recognized and mitigated in the development grading. Although the majority of the bedrock is not considered to be unusually prone to sliding, clay-rich beds within the bedrock may be susceptible to sliding, particularly if erosion or excavations have undercut the strata, leaving the slope without sufficient lateral support.





MAP OF FAULTS AND REGIONAL SEISMICITY
OF

LAWRENCE PROPERTY
(FROM MORTON AND OTHERS, 1974)



TABLE 1
SEISMIC PARAMETERS FOR
LAWRENCE PROPERTY
ORANGE COUNTY, CALIFORNIA

POTENTIAL CAUSATIVE EARTHQUAKE FAULT	CLOSEST DISTANCE FROM FAULT TO SITE	LENGTH OF FAULT (NOTE 1)	RICHTER MAGNITUDE OF HISTORICAL EARTHQUAKE	APPROXIMATE AGE OF MOST RECENT SURFACE DISPLACEMENT	MAXIMUM CREDIBLE EVENT		MAXIMUM PROBABLE EARTHQUAKE (DESIGN EARTHQUAKE)			
					RICHTER MAGNITUDE	MEAN ESTIMATED RECURRENT INTERVAL IN YEARS (NOTE 2)	RICHTER MAGNITUDE	PEAK HORIZONTAL GROUND ACCELERATION AT SITE (FRACTION OF GRAVITY) (NOTE 4)	PREDOMINATE PERIOD AT SITE (IN SECONDS) (NOTE 5)	DURATION OF STRONG SHAKING AT SITE (IN SECONDS) (NOTE 6)
ANDREAS LT (SOUTH GARLOCK LT)	61 km	500 km	8.25+ (1857)	HISTORIC (1857 and 1948)	8.5	M8=125-225 (NOTE 7)	8.3	.21	0.48	23
	38 mi	310 mi	6.5 (1948)							
JACINTO LT	50 km	440 km	7.0 (1899)	HISTORIC (1899 and 1968)	7.5	M6 = 4-10 M7 = 40-100 M8 = 400-1000	7.2	.15	0.35	24
	31 mi	274 mi	6.5 (1968)							
TIER- INORE- A CALIENTE LT	11 km	260 km	5.5 (1938)	HISTORIC (1910)	7.5	M6 = 20-90 M7 = 200-900 M8 = 3000-9000	6.7	.41 (.27)*	0.30	22
	7 mi	162 mi	6.0± (1910)							
RRA MADRE LT	47 km	90 km	6.4 (1971)	HISTORIC (1971)	7.0	M7 = 5000 (NOTE 3)	6.5 (NOTE 3)	.10	0.30	14
	29 mi	56 mi								
PORT- EWOOD LT	27 km	80(+) km	6.3 (1933)	HISTORIC (1933, unconfirmed)	7.0	M7 = >300(?)	6.5	.20 (.13)*	0.22	15
	17 mi	50(+) mi								

Note 1.

Postulated maximum rupture length based on L/2
(maximum credible earthquake), and L/5
(maximum probable earthquake).

Note 2.

After D. Lamar, 1973.

Note 3.

After R. Crook, B. Kamb, C. Allen, M. Payne
and R. Proctor, 1978.

Note 4.

After Schnabel and Seed, 1973. The repeatable
high ground acceleration (*), taken as 65 percent
of the peak acceleration, for sites within 20+ miles
of the epicenter (after Ploessel and Slosson, 1974),
may be more applicable for design analysis.

Note 5.

After Seed, Idriss and Kiefer (1969).

Note 6.

After Bolt (1973).

Note 7.

After Sieh (1981), for "south central section"
of fault near Cajon Pass.

4.0 RESOURCES

4.1 Groundwater and Surface Runoff

The site is in the upper part of the Aliso Creek watershed, which is approximately 15 miles from the ocean where the creek empties. Considering the relatively rapid runoff and probably minimal percolation which is typical of this terrain, groundwater resources within the property are most likely limited to perched water zones within the alluvium and saturated bedrock zones immediately beneath the alluvium. Although no runoff was observed in the creek at the time of our reconnaissance, it and other drainage courses carry seasonal flows. An old water well with a windmill next to the northerly dwelling reportedly provides small amounts of water for irrigation purposes. We understand that there is a spring on the hillside above the property.

The principal groundwater resources within the Aliso Creek watershed are located about 3 miles south, along El Toro Road, where the alluvium is more extensive. There, water well records indicate that water levels range from about 5 to 25 feet from the surface in past years, with a few being dry in the summer months (Reference 2).

4.2 Mineral Deposits

According to the California Division of Mines and Geology (Reference 9) the nearest sites of economically significant mineral resources are located across Santiago Canyon Road, about ½ mile north of the subject property. There, two mines (the Serrano Mine, and later the Schoeppe Mine) were in operation intermittently from 1926 to 1975, when all activity was ceased. Both produced clay and silica sand from the upper part of the Silverado Formation. This formation does not underlie the subject property, and there is no known mineral resources of economic importance contained in the formation which is present.



5.0 PRINCIPAL GEOTECHNICAL HAZARDS, CONSTRAINTS, IMPACTS AND POSSIBLE MITIGATION MEASURES

5.1 Factors or Geologic Problems Evaluated

This section presents, in summary form, the principal geotechnical factors that were considered and rated on a subjective scale, comparing the subject site with the range of hazard severity which is generally representative in southern California, refer to Table 2, which presents, in matrix form, the hazard rating and possible mitigative measures). A discussion of the principal geologic and hydrogeologic constraints follows.

5.2 Fault Displacement

In the absence of any major faults crossing the site, the hazard of ground rupture from fault displacement is considered to be nil. Therefore, no special fault studies or construction limitations due to fault movement potential apply to the subject site. Subsequent geologic site investigations and inspections required by the County at various development stages should provide appropriate mitigation measures if a major fault is found within the site.

5.3 Ground Shaking

Although moderate intensities of seismic ground shaking can be anticipated at the site (either from smaller earthquakes located relatively close, or from larger earthquakes farther from the site), the effects are expected to be satisfactorily mitigated by conformance with the latest (1982) Uniform Building Code, the Orange County Building Code, or recommendations of the Structural Engineers Association of California for seismically resistive design of structures.

5.4 Liquefaction and Related Ground Failure Phenomena

Secondary earthquake hazards, such as liquefaction, flow landsliding, seismically induced settlement, and ground lurching or cracking (shown as "ground rupture" in Table 2) are generally associated with relatively high intensities of ground shaking, shallow groundwater conditions and the presence of loose sandy soils or alluvial deposits. Because of probable soil conditions which underlie most of the site, and taking into account the moderate ground shaking intensities which could occur, such ground failure hazards are rated as slight, even though shallow groundwater could be present locally (along the canyon bottom) during and shortly after the rainy seasons. Therefore, no special mitigation measures, other than construction in accordance with code requirements is expected to be necessary. The detailed soil investigation required at the parcel map or building plan stage should provide sufficient data to permit a more definitive evaluation of the hazard.



LAWRENCE PROPERTY

TABLE 2. CHECKLIST OF GEOTECHNICAL HAZARDS AND POTENTIAL MITIGATION MEASURES
(MODIFIED FROM CDMG NOTE 46)

G E O L O G I C P R O B L E M S		. DEGREE OF HAZARD OR PROBLEM				POSSIBLE MITIGATION MEASURES		
PROBLEM	ACTIVITY CAUSING PROBLEM	NOISE	SLIGHT	MODERATE	SEVERE	CODE CONFORMANCE	CODE CONFOR- MANCE + SPECIAL WORK*	ADVANCE PLAN- NING, AVOIDANCE, RESTRICTIONS
EARTHQUAKE DAMAGE	FAULT MOVEMENT	X	X			X		
	LIQUEFACTION	X	X			X		
	LANDSLIDES		X			X		
	DIFFERENTIAL COMPACTION/ SEISMIC SETTLEMENT	X	X			X		
	GROUND RUPTURE		X				X	
	GROUND SHAKING		X	X		X		
	TSUNAMI	X				N.A.		
	SEICHES	X				N.A.		
	FLOODING (DAM OR LEVEE FAILURE)	X	X			N.A.		
LOSS OF MINERAL RESOURCES	LOSS OF ACCESS		X			X		
	DEPOSITS COVERED BY CHANGED LAND USE		X			X		
	ZONING RESTRICTIONS		X			X		
WASTE DISPOSAL PROBLEMS	CHANGE IN GROUNDWATER LEVEL		X					X
	DISPOSAL OF EXCAVATED MATERIAL		X			X		
	PERCOLATION OF WASTE MATERIAL		X	X			X or	X
SLOPE AND/OR FOUNDATION INSTABILITY	LANDSLIDES AND MUDFLOWS		X	X			X	
	UNSTABLE CUT AND FILL SLOPES		X				X	
	COLLAPSIBLE AND EXPANSIVE SOIL		X	X		X		
	TRENCH-WALL STABILITY		X			X		
EROSION, SEDIMENTA- TION, FLOODING	EROSION OF GRADED AREAS			X		X		
	ALTERATION OF RUNOFF		X				X	
	UNPROTECTED DRAINAGE WAYS		X	X			X	
	INCREASED IMPERVIOUS SURFACES		X			X		
LAND SUBSIDENCE	EXTRACTION OF GROUNDWATER, GAS, OIL, GEOTHERMAL ENERGY	X	X			X		
	HYDROCOMPACTION, PEAT OXIDATION	X				X		
VOLCANIC HAZARDS	LAVA FLOW	X				N.A.		
	ASH FALL	X				N.A.		

*"SPECIAL WORK" CAN INCLUDE ADDITIONAL INVESTIGATION, SPECIAL SITE PREPARATION, OR SPECIAL FOUNDATIONS.



5.5 Tsunami, Seiche, and Inundation Hazard

No problems connected with tsunamis (seismically generated sea waves), seiches (seismically generated waves in lakes or reservoirs), or inundation caused by dam failure are expected to affect the site. The possible future pond in the tributary canyon northwest of the main house would not appear to pose a significant hazard if appropriately designed and constructed.

5.6 Slope Instability

The formations underlying the site have been grossly stable, as judged by the absence of landslides. But because of the moderate to steep topography which characterizes the west portion of the site, and the presence of surficial soil and weathered bedrock which may, in part, be susceptible to shallow failures (such as mudflows), careful analysis of future development plans will be necessary to appropriately mitigate potential slope or building site stability problems. This could require corrective measures if unstable materials would be exposed by grading, or underlie the existing slopes adjoining proposed improvements. Possible alternatives to such standard mitigation measures as buttress fills or retaining walls would be to modify proposed slopes to have shallower gradients, lower heights, or by reorientation of slope direction. Structures also could be relocated away from hazardous slopes, particularly if they cannot be stabilized feasibly.

5.7 Flooding, Erosion and Sedimentation

The hazard of flooding, along with erosion and sedimentation, resulting from storm runoff is rated as slight for most of the property, although these could range to moderate or severe along the main drainage courses during heavy runoff periods. According to the National Flood Insurance Program maps (Reference 12), the potential inundation area anticipated from a 100-year flood in Aliso Creek begins at Cook's Corner and extends southward, away from the subject site. The existing Parcel Map for the property (Reference 4), however, indicates a flood hazard and inundation zone along Aliso Creek, presumably representing a more frequent flood recurrence interval delineated in accordance with Orange County criteria. Appropriate mitigation for such flood hazards would be to locate all habitable structures outside of any delineated zone or other drainage course.

On graded slopes, the erosion and sedimentation potential should be slight if they are constructed and landscaped in accordance with code requirements. Adequate surface drainage and control devices will be necessary to appropriately mitigate the potential runoff, erosion and sedimentation problems adjoining and within the sites proposed for development.

5.8 On-Site Sewage Disposal

The principal constraints for the use of on-site sewage disposal systems will be the highly variable soil or bedrock formation permeabilities likely to be encountered within the site, and the topographic limitations of the steeper terrain. These constraints, and potential negative impacts (such as reduction of slope stability from subsurface infiltration of sewage effluent) can be overcome or mitigated by adequate investigation of the site and appropriate design of a system on a site-by-



site basis. We understand that the existing onsite sewage disposal systems (septic tank and leach lines) have performed satisfactorily. Future building sites on the hillside, where the soils are likely to be less permeable, may require larger or a different type of sewage disposal system.

Considering the relatively low housing density likely to be proposed, the groundwater degradation impact of private sewage disposal systems is expected to be minimal as regards the main groundwater resources located several miles downstream. However, their impact on the local groundwater in the canyon bottom could be more significant.

5.9 Other Hazards, Constraints, or Impacts

All other potential adverse factors affecting the site (listed on Table 2) either do not apply or are expected to be of no more than average severity and mitigable by use of designs, standards or procedures prescribed by applicable codes.



APPENDIX A



APPENDIX A

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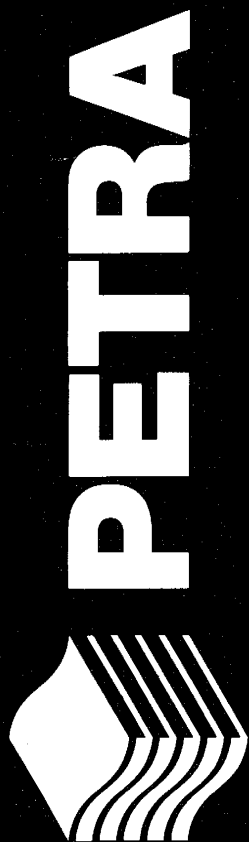
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AERIAL PHOTOS REVIEWED

<u>Year</u>	<u>Flight</u>	<u>Photo No.</u>	<u>Source</u>
1939	5925	69, 74	Fairchild
1983	82117	3, 4	Don Read





**GEOTECHNICAL INVESTIGATION, PROPOSED
BUILDING A-D, COVERED BRIDGE, ENTRANCE
GATE, AND EXISTING BUILDING "E", RANCHO
LAS LOMAS, 19191 LAWRENCE CANYON ROAD,
COUNTY OF ORANGE, CALIFORNIA**

MR. AND MRS. RICK LAWRENCE

**November 9, 1995
J.N. 147-95**

November 9, 1995
J.N. 147-95

MR. AND MRS. RICK LAWRENCE
c/o Mr. James R. Walton
1858 Hunkpapa Street
S. Lake Tahoe, CA 96150

Subject: Geotechnical Investigation, Proposed Building A-D, Covered Bridge, Entrance Gate, and Existing Building "E", Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California.

References: See attached list.

Dear Mr. and Mrs. Lawrence:

We are pleased to submit herewith our geotechnical investigation report for proposed Building A-D, a covered bridge, an entrance gate and existing Building "E" located within the grounds of Rancho Las Lomas at 19191 Lawrence Canyon Road in the County of Orange. This work was performed in accordance with the scope of work outlined in our Proposal dated July 19, 1995. This report presents the results of our field investigation, laboratory testing and our engineering judgement, opinions, conclusions and recommendations pertaining to the design and construction of proposed Building A-D, a covered bridge, an entrance gate, and to the existing geotechnical conditions beneath building "E".

Several members of our professional staff participated in this geotechnical investigation. The field investigation was performed by Mr. David Hansen, Senior Staff Engineer, who also coordinated other project activities. Selection of laboratory tests, foundation engineering analyses, and development of geotechnical recommendations were performed by Mr. Charles Byrd, Associate Engineer.

MR. AND MRS. RICK LAWRENCE

November 9, 1995

J.N. 147-95

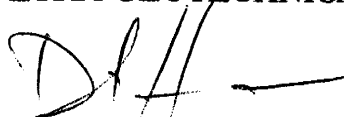
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Dr. Siamak Jafroudi, the undersigned Principal Engineer and Mr. Robert Ruff, Principal Geologist, provided technical supervision and direction as needed throughout the entire investigation and assisted with the preparation of this report.

It has been a pleasure to be of service to you on this project. Should you have any questions regarding the contents of this report, or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

PETRA GEOTECHNICAL, INC.


for Dr. Siamak Jafroudi
Principal Engineer

SJ/nls



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APPENDIX C

Log of Previous Test Pits
(Strata-Tech)



**GEOTECHNICAL INVESTIGATION, PROPOSED BUILDING A-D,
COVERED BRIDGE, ENTRANCE GATE, AND EXISTING BUILDING
"E", RANCHO LAS LOMAS, 19191 LAWRENCE CANYON ROAD,
COUNTY OF ORANGE, CALIFORNIA**

INTRODUCTION

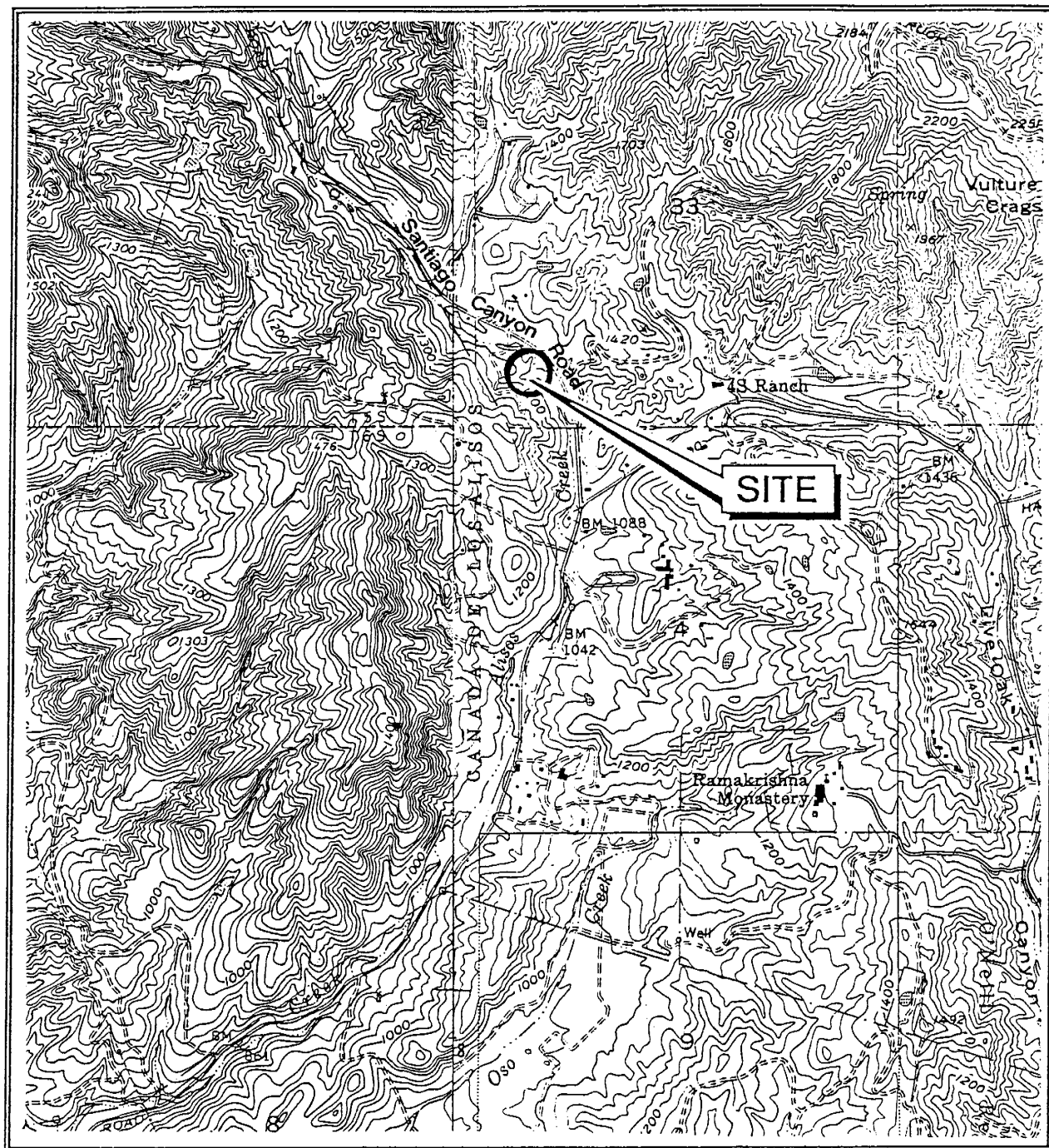
This report presents the results of our geotechnical investigation for proposed Building A-D, a covered bridge, an entrance gate and for existing Building "E" located within the grounds of Rancho Las Lomas. The purposes of this investigation were to determine the nature of surface and subsurface soil conditions beneath the proposed and existing buildings, to evaluate their in-place characteristics, to evaluate the suitability of the foundation soils for support of the buildings and to provide geotechnical recommendations for design and construction of the new buildings and for mitigation of any conditions not complying with the County of Orange Grading Code for existing Building "E".

This study included a review of previous reports prepared for other portions of the property and a review of available published and unpublished literature and geotechnical maps with respect to active and potentially active faults located in proximity to the site which may have an impact on the proposed construction.

LOCATION AND DESCRIPTION

Proposed Building A-D, a covered bridge, an entrance gate and existing Building "E" are located (or will be located) within the grounds of Rancho Las Lomas which is located at 19191 Lawrence Canyon Road in the County of Orange. The general location of the site is shown on Figure 1. Rancho Las Lomas is located on the easterly facing flank of a north-south-trending ridge. Maximum topographic relief is approximately 231 feet, ranging from a high of 1346 feet above sea level near the southwestern corner of the property to a low of 1115 feet





LOCATION MAP

*Ref: U.S.G.S. 7.5 Minute Series Topographic Maps;
 SANTIAGO PEAK QUADRANGLE (1954, Revised 1988)
 and EL TORO QUADRANGLE (1968)
 SCALE: 1 inch = 2000 feet*



above sea level within the southeastern corner. The subject property is bordered on the north and south by large residential estates, on the west by a residential tract, and on the east by Santiago Canyon Road. A site plan (Plate 1) is presented in the map pocket at the end of this report.

The buildings which are the subject of this report are located (or will be located) within the eastern portion of the site near Santiago Canyon Road. As mentioned above, most of the site consists of an easterly facing slope; however, the eastern portion is relatively level with the exception of Aliso Creek which has created a small, meandering channel. A manmade earth berm also borders Santiago Canyon Road.

Building A-D is proposed near the southern end of the site approximately 30 feet away from Aliso Creek and adjacent to the earth berm. Existing Building "E" is located within the south-central portion of the site in proximity to Aliso Creek. The covered bridge, designated as Building "Y" on the enclosed site plan (Plate 1), is proposed within the central eastern portion of the site and will span Aliso Creek. The entrance gate is proposed at the northern end of the property in proximity to Santiago Canyon Road.

PROPOSED CONSTRUCTION AND GRADING

It is proposed to construct a two-story building (Building A-D), a covered bridge, and an entrance gate within the subject property. In addition, remedial work will be performed on an existing building (Building "E") in order to bring it into compliance with the Codes of the County of Orange Building Department.



The two-story building (Building A-D) will be of both woodframe and masonry block construction with the first floor slab constructed on-grade. The exterior wall on the eastern side of the building will be constructed as a retaining wall since it will extend into the base of the existing earth berm (see Cross Section C-C', Plate 2). The earth berm at this location is approximately 15 feet high with a side slope gradient of approximately 2:1, horizontal to vertical.

The covered bridge will be approximately 45 feet long by 21 feet wide and of woodframe construction. Although specific foundation plans for the bridge have not been prepared at this time, it is expected that the eastern end of the bridge will be supported on a small bridge abutment while the western end of the bridge will be supported on conventional footings. The small bridge abutment will most likely consist of several connected retaining walls.

The entrance gate will consist of two large wood gates supported by two masonry block pilasters. A wood frame arch and tower will be constructed above the wood doors and will also be supported by the two pilasters. In addition, masonry block walls and raised planters will be constructed on either side of the entrance gate.

Although definitive grading plans are not available for review, it is anticipated that finish grades within each proposed building site will generally correspond to existing elevations. The majority of the grading will consist of minor cuts and fills of only a few inches; however, deeper cuts will be required along the eastern side of proposed Building A-D in order to construct the retaining wall along the base of the earth berm. The only significant fill to be placed will be behind the retaining wall of Building A-D and within the abutment area for the covered bridge. In addition, a certain amount of remedial grading involving the recomp-



action of surface soils will be necessary in local areas to achieve proper support of the proposed structures. Recommendations for general site grading and remedial grading are given in the "Conclusions and Recommendations" section of this report.

SITE HISTORY

Numerous previous geotechnical investigations have been performed by both Action Geotechnical and Strata-Tech to evaluate geotechnical conditions within the site. The two most significant reports with respect to this current investigation were prepared by Strata-Tech. They are significant due to the fact that they included subsurface work in relative close proximity to the proposed buildings and existing Building "E". The first report, dated August 1, 1990, was prepared to evaluate the condition of the buildings and underlying soils in Phase I (Reference No. 4). This phase included buildings S, T, U, V, X, Y and Z. The second report, dated January 21, 1992, was prepared for Phase II which included buildings A through G, J through N, and P through R (Reference No. 10). Building "E", which was investigated as a part of the Phase II investigation, was also investigated as part of this report.

Our firm has performed two previous geotechnical investigations within the site. The first investigation was for buildings "H", "A-A", and "A-B" (Reference No. 23). This previous investigation included the excavation of eight exploratory test pits (Test Pits TP-1 through TP-8) and one exploratory boring (Boring B-1). The second investigation was for the proposed greenhouse, tool storage room, restrooms, and existing Building "C" (Reference No. 24). This investigation included the excavation of one exploratory test pit (Test Pit TP-9) and the drilling of ten hand-augered exploratory borings (Borings B-2 through B-11).



FIELD EXPLORATION

Subsurface exploration for this report included the drilling of eight hand-augered exploratory borings (Borings B-12 through B-18) and one hand-excavated exploratory test pit (Test Pit TP-10). The exploratory borings were drilled throughout the areas of proposed construction in order to determine the subsurface soil conditions. The test pit was excavated beneath the eastern corner of building "E" in order to expose the existing footings and underlying soils. The exploratory borings were drilled to depths of 8 to 15 feet with hand-augering equipment. The exploratory test pit was excavated to a depth of 3 feet using a pick and shovel. Earth materials encountered were classified and logged in accordance with the visual-manual procedures of the Unified Soil Classification System. Representative bulk samples of soil were also obtained for laboratory testing. Relatively undisturbed samples of subsurface soils were obtained with a hand-driven, split-spoon sampler. The approximate boring and test pit locations are shown on the accompanying site plan (Plate 1) and descriptive "Exploration Logs" and "Test Pit Logs" are presented in Appendix A.

In addition, several pertinent previous test pits that were excavated by Strata-Tech as part of their investigations for Phases I and II are shown on the enclosed plan. The logs of these test pits are presented in Appendix C.

LABORATORY TESTING

To further evaluate the engineering properties of site soils, laboratory tests were performed on selected samples considered representative of those encountered. Laboratory tests included the determination of maximum dry density, expansion potential, soluble sulfate content, consolidation potential and shear strength characteristics. The results of these tests as well as a description of laboratory



test criteria are presented in Appendix B. Moisture content and unit dry density of the in-place soils were also determined in representative strata and are presented in Appendix B. An evaluation of the test data is reflected throughout the "Conclusions and Recommendations" section of this report.

FINDINGS

Regional Geologic Setting

Rancho Las Lomas is located on the southwestern flank of the Santa Ana Mountains in eastern Orange County. Furthermore, the property is located along the extreme eastern margin of the Capistrano Embayment within the Peninsular Ranges Geomorphic province of California. The Capistrano Embayment is a generally north-south trending structural feature comprised of a thick sequence of Tertiary age sedimentary bedrock gently folded into a broad syncline. This embayment has been structurally down-dropped to the west and southwest of the Santa Ana Mountains along several faults including the Aliso, Mission Viejo and Christianitos Faults.

The general area is characterized by narrow bands of Eocene to Miocene age sedimentary rocks that generally strike northerly to northwesterly and dip to the west-southwest at gentle to moderate inclinations. These bands of tertiary-aged rock have locally formed hogback ridges and narrow parallel valleys of low to moderate relief.

Local Geology

Based on our exploratory borings and test pit, the areas of proposed construction are underlain by colluvial materials. Although not encountered during this exploratory investigation, bedrock materials belonging to the Sespe Formation



underlie the site at depth. These bedrock materials were encountered within the higher portions of the property during the previous exploratory investigations by our firm, Strata-Tech and Action Geotechnical. Regional geologic mapping and local geologic mapping by both Strata-Tech and Petra Geotechnical during previous investigations indicate that the bedrock within the site dips 35 to 40 degree towards the west.

The colluvial materials consist of a variety of different soil materials that vary greatly in their in-place densities and consolidation characteristics from one location to the next. The localized conditions of the colluvial materials beneath the structures are addressed in the "Local Geotechnical Conditions" section of this report.

Groundwater

Groundwater was not encountered within our exploratory borings or test pit, at least to the maximum depth explored (15 feet).

Faulting

Based on our review of published and unpublished geotechnical maps and literature pertaining to site and regional geology, the closest active fault to the site is the Elsinore Fault. This fault, which lies approximately 10 miles to the north-east of the site, consists of en-echelon, northwest to west-northwest trending faults. Many of these faults are accompanied by geomorphic and geologic features that indicate they have been active during late Quaternary and Holocene time. Five historic earthquakes with magnitudes exceeding 5 are known to have



retaining wall and constructed to such a height that the requirements of setback "f" as shown on Figure 2 of the County of Orange Grading Manual. A copy of Figure 2 is attached for convenience.

Based on our exploratory borings, the colluvial materials beneath proposed Building A-D consist of approximately 10 feet of sandy silt underlain by silty sand. The sandy silt was observed to be moist and firm but porous in the upper 3 feet, and then moist and stiff below with only a trace of porosity. The underlying silty sand was observed to be moist, dense and fine grained with trace amounts of gravel.

Consolidation test data indicate that the materials within the upper 3 feet of this building area are susceptible to a moderate degree of collapse potential (hydro-consolidation) under saturated conditions while the underlying materials are not subject to collapse. The collapse potential of the sample obtained at a depth of 2 feet was approximately 1.8 percent under existing overburden pressures while the sample obtained at a depth of 6 feet did not show any collapse. Geo-technical interpretation of our consolidation test results indicates that a future post-construction settlement of approximately 0.7 inches could possibly occur below the building if no remedial measures are taken.

Existing Building E

This single-story building is of woodframe construction with the floor slab constructed on-grade. The slope above Building "E" has a gradient of less than 5:1 and therefore, no specific structural setback is required. However, the northeastern side of the building is located within 5 feet of the top of the adjacent 12-foot-high slope that descends down to Aliso Creek. This setback



occurred along the Elsinore Fault. Three occurred near Lake Elsinore in 1910, with the largest having an estimated magnitude of 6.0 (Topozada, Parke, and Higgins, 1978). A number of smaller shocks (magnitude 4.0 to 5.0) with instrumentally determined epicenters have been reported along the fault (Real, Topozada, and Parke, 1978). Displacement of Holocene-age sediments and evidence for ground surface rupture associated with the 1910 earthquake have also been documented in a trench along the Glen Ivy Fault, a branch of the Elsinore fault zone (Rockwell and others, 1985).

No other active or potentially active faults project through or toward the site, nor does the site lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act.

CONCLUSIONS AND RECOMMENDATIONS

General

From a soils engineering and engineering geologic point of view, the subject property and the subsurface conditions are considered suitable for the subject buildings provided the following recommendations are implemented in order to bring the buildings into compliance with prudent geotechnical engineering practices and the requirements of the County of Orange.

Geotechnical Conditions

Proposed Building A-D

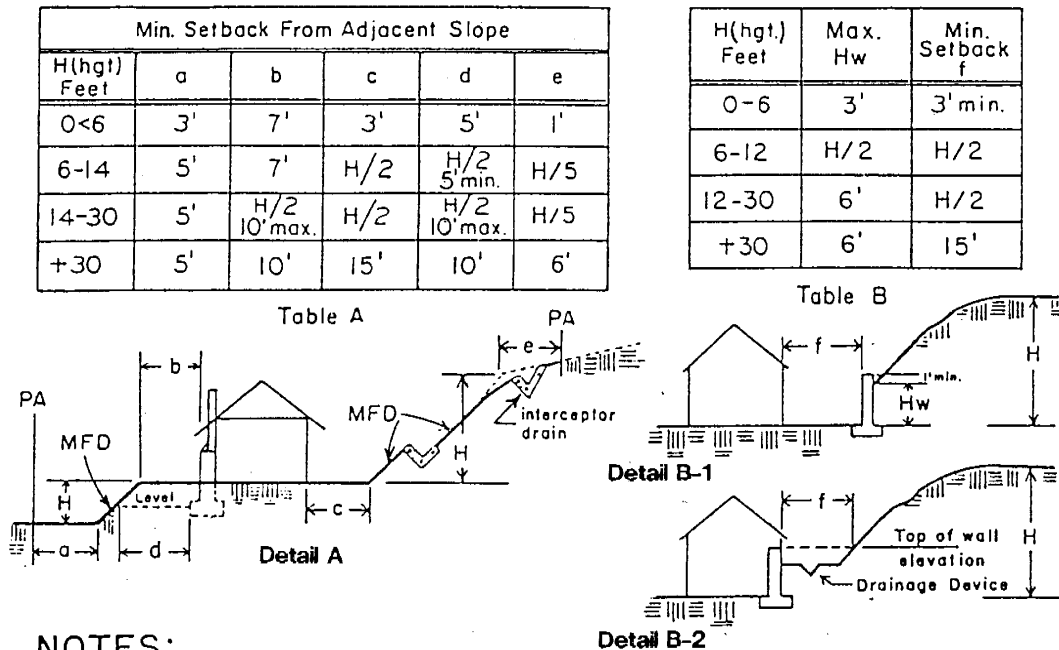
This building will be constructed into the base of an approximately 15-foot-high earth berm. The rear wall of the building will need to be constructed as a



SETBACK REQUIREMENTS

(County of Orange Grading Manual)

Figure 2



NOTES:

1. PA means permit area boundary and/or property line; MFD means manufactured surface.
2. Setbacks shall also comply with applicable zoning regulations.
3. Table A applies to manufactured slopes and 2:1 or steeper natural slopes. Setbacks from natural slopes flatter than 2:1 shall meet the approval of the Building Official.
4. "b" may be reduced to 5' minimum if an approved drainage device is used; roof gutters and downspouts may be required.
5. "b" may be reduced to less than 5' if no drainage is carried on this side and if roof gutters are included.
6. If the slope between "a" and "b" levels is replaced by a retaining wall, "a" may be reduced to zero and "b" remains as shown in Table A. The height of the retaining wall shall be controlled by zoning regulations.
7. "b" is measured from the face of the structure to the top of the slope.
8. "d" is measured from the lower outside edge of the footing along a horizontal line to the face of the slope. Under special circumstances "d" may be reduced as recommended in the approved soil report and approved by the Building Official.
9. The use of retaining walls to reduce setbacks (Fig. B) must be approved by the Building Official.
10. "f" may be reduced if the slope is composed of sound rock that is not likely to produce detritus and is recommended by the soil engineer or engineering geologist and approved by the Building Official.
11. "a" and "e" shall be 2' when PA coincides with Arterial or local street right of way and when improved sidewalk is adjacent to right of way.
12. "e" shall be increased as necessary for interceptor drains.

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does not meet the requirements of either setback "b" or "d" of Figure 2. In addition, the adjacent creek channel is subject to erosion during periods of heavy water runoff and could potentially undermine the footing.

Our exploratory test pit and a previous test pit by Strata-Tech indicate that building "E" is supported on very shallow footings (6 to 12 inches deep). The building is underlain by colluvial materials that extend to a depth of at least 10 feet below the ground surface. The depth to bedrock could not be determined due to the presence of gravel and cobbles within the colluvial materials that prevented the boring from being advanced beyond a depth of 10 feet.

The colluvial materials beneath existing Building "E" consist of approximately 7 feet of clayey sand underlain by silty sand. The clayey sand materials were observed to be moist but porous and medium dense to dense in the upper 4 feet, and then moist and dense below. The silty sand materials were observed to be moist, dense and fine- to medium-grained with abundant gravel and frequent cobbles. Consolidation test data indicate that the colluvial materials are susceptible to a moderate to very low degree of collapse potential (hydroconsolidation) under saturated conditions. The collapse potential of the samples tested varied from approximately 1.9 percent for the colluvial materials within the upper 4 feet of the ground surface to 0.15 percent for the lower colluvial materials under existing overburden pressures. Geotechnical interpretation of our consolidation test results indicates that a future post-construction settlement of approximately 1 inch could possibly occur below building "E" when the colluvial materials become saturated.



Proposed Covered Bridge

The footings for the proposed bridge will be located on or at the top of the slopes that descend down to Aliso Creek. The footings for the proposed bridge will need to be deepened in order to meet the setback requirements of Figure 2 of the County of Orange Grading Manual. In addition, the creek channel is subject to erosion during periods of heavy water runoff and could potentially undermine the bridge footings.

The proposed bridge will be underlain by colluvial materials that range up to at least 11 feet in depth. The colluvial materials within the area of the bridge were found to have highly variable densities and consolidation characteristics. The colluvial materials range from medium dense to dense and are locally slightly porous to porous. The colluvial materials also contain highly variable amounts of gravel and cobbles. Consolidation test data indicate that the colluvial materials are susceptible to a low to moderate degree of collapse potential (hydroconsolidation) under saturated conditions. The collapse potential of the samples tested varied from approximately 1.2 to 2.8 percent for the colluvial materials within the upper 7 feet and varied from 0.2 to 0.7 percent for the deeper colluvial materials under existing overburden pressures. Geotechnical interpretation of our consolidation test results indicates that a future post-construction settlement of approximately 1.5 inches could possibly occur below the bridge when the colluvial materials become saturated.

Proposed Entrance Gate

The entrance gate will be located on level ground; therefore, structural setbacks are not a concern.



The proposed entrance gate will be underlain by approximately 11 feet of colluvial materials. The colluvial materials are underlain by sandstone bedrock materials of the Sespe Formation. The colluvial materials beneath the proposed entrance gate were found to be medium dense and locally porous within the upper 3 feet, and moist and dense with only slight porosity below. Consolidation test data indicate that the colluvial materials are susceptible to a low to very low degree of collapse potential (hydroconsolidation) under saturated conditions. The collapse potential of the samples tested varied from approximately 1.3 percent for the colluvial materials within the upper 3 feet to 0.2 percent for the lower colluvial materials under existing overburden pressures. Geotechnical interpretation of our consolidation test results indicates that a future post-construction settlement of approximately 0.8 inches could possibly occur below the entrance gate when the colluvial materials become saturated.

Recommendations for Proposed Building A-D

Pad Grading

This building site is underlain by colluvial materials that are at least 11 feet deep and are subject to a moderate degree of hydroconsolidation within the upper 3 feet. Therefore, in order to mitigate distress related to the potential adverse affects of excessive total and differential settlements, the existing ground surface to a depth of 3 feet should be removed and recompactd. Recommendations for this remedial grading are presented in the "Earthwork" section of this report.

Allowable Bearing Capacity

If remedial grading is performed as described above, an allowable bearing value of 1500 pounds per square foot may be used for design of 12-inch-wide footings founded at a minimum depth of 12 inches into competent fill materials. This



bearing value may be increased by 20 percent for each additional foot of depth, to a maximum value of 2500 pounds per square foot. The allowable bearing value may be increased by one-third when designing for short duration wind and seismic forces.

Lateral Resistance

A passive earth pressure of 250 pounds per square foot per foot of depth may be used to determine lateral bearing for footings. A coefficient of friction of 0.35 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for short duration wind and seismic forces. The above values are based on footings placed directly against compacted fill. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of maximum dry density.

Settlement

If remedial grading is performed as recommended above, the upper 3 feet of porous and collapsible soils will be removed and replaced with compacted fill. Calculations indicate that the total maximum settlement of footings underlain with compacted fill will be approximately $\frac{1}{2}$ of an inch and the maximum differential settlement will be approximately $\frac{1}{4}$ of an inch over a span of approximately 30 feet. The majority of the above footing settlements will occur during construction as building loads are applied.



Setbacks

The easterly wall of the Building A-D will should be constructed to such a height that the requirements of setback "f" as shown on Figure 2 will be met. For a slope height of 15 feet, a horizontal setback of 7.5 feet will be required between the top of the wall and the face of the slope.

Expansive Soil Considerations

The results of our laboratory tests indicate that the soil materials existing within the site exhibit a Low to Medium expansion potentials as classified in accordance with UBC Table No. 29-C. The design and construction details presented below may be considered for minimizing the effects of expansive soils. These recommendations have been developed based on the previous experience of this firm on projects with similar conditions. Construction performed in accordance with these recommendations has been found to minimize but not positively prevent post-construction movement, cracking, and other effects of expansive soils. The owner, architect, design civil engineer, structural engineer, and contractors must be made aware of the expansive soil conditions which exist on the site. Additional slab thickness, footing size and reinforcement should be provided as required by the structural engineer.

1. All exterior footings should be founded at a minimum depth of 18 inches below the lowest adjacent final grade. Interior footings may be founded at a minimum depth of 12 inches below the lowest adjacent final grade.
2. All continuous footings should be reinforced with two No. 4 bars, one top and one bottom.



3. Isolated pad footings should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced 18 inches on centers, both ways, near the bottoms of the footings.
4. Living area concrete floor slabs should be a full 4 inches thick, and reinforced with 6-inch by 6-inch, No. 10 by No. 10 welded wire mesh, or with No. 3 bars spaced 24 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid-depth.
5. Living area floor slabs should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 6-mil Visqueen, or equivalent. At least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete.
6. Garage floor slabs should be a full 4 inches thick and reinforced in a similar manner as living area slabs. Garage floor slabs should also be poured separately from adjacent wall footings with a positive separation maintained with 3/8-inch minimum felt expansion joint materials, and then quartered with weakened plane joints. A 12-inch-wide by 18-inch-deep grade beam should be provided across garage entrances. The grade beam should be reinforced with two No. 4 bars, one top and one bottom.
8. Prior to placing concrete, the subgrade soils below all living area and garage floor slabs should be presoaked to achieve a moisture content that is 5 percent or greater above optimum moisture content. This moisture content should penetrate to a minimum depth of 18 inches into the subgrade.



Active and At-Rest Earth Pressures for Retaining Wall Design

For a cantilevered wall condition, active earth pressures equivalent to a fluid having a density of 45 pounds per cubic foot for a level backfill and 75 pounds per cubic foot for an ascending 2:1 backfill are recommended for design of the easterly building wall. If the wall is restrained at the top, at-rest earth pressures equivalent to a fluid having densities of 68 and 110 pounds per cubic foot are recommended for a level backfill and ascending 2:1 backfill, respectively. The above values are for a well-drained backfill. All walls should also be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above active and at-rest earth pressures.

Temporary Excavations for Easterly Wall

The fill materials underlying the earth berm consist of slightly cohesive silty sands and clayey sands that are moderately to well-compacted. For this type of material, temporary excavations to accommodate construction of the easterly wall that are equal to or less than 6 feet in height may be cut vertical. For excavations which exceed a height of 6 feet, the lower 6 feet may be cut vertical, and the upper portions exceeding this height should be cut back at a maximum gradient of 1:1, horizontal to vertical. However, all temporary slopes should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending on the results of these observations, flatter temporary slopes may be necessary.

Drainage and Moisture-Proofing

A perforated pipe and gravel subdrain should be installed behind the wall to prevent entrapment of water in the backfill. Perforated pipe should consist of 4-inch minimum diameter ABS SDR-35 or PVC Schedule 40 with the perfora-



tions laid down. The pipe should be embedded in $\frac{3}{4}$ - to $1\frac{1}{2}$ -inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend to a minimum height of 2 feet above the footing. Filter fabric should consist of Mirafi 140N, or equivalent. A solid outlet pipe should be connected to the subdrain and then routed to a suitable area for discharge of accumulated water. The outside portions of the wall supporting backfill should also be coated with an approved waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

The recommended active and at-rest earth pressures are based on the physical and mechanical properties of the onsite soils and they may be used for wall backfill. All wall backfill should be placed in 6- to 8-inch-thick maximum layers and mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting should not be permitted. Probing and testing should be performed by a representative the project geotechnical consultant to verify adequate compaction of the backfill.

Recommendations for Existing Building "E"

Underpinning

Building "E" lacks a adequate foundation and is supported by colluvial materials that are subject to hydroconsolidation, especially within the upper 4 feet. Therefore, this building should be underpinned with new continuous footings, or with circular pier footings and grade beams. New footings should be embedded at least 24 inches below the lowest adjacent finish grade.



Setbacks

The structural setback between the northeasterly side of Building "E" and the adjacent descending slope does not meet the requirements of setbacks "b" and "d" shown on Figure 2 of the County of Orange Grading Manual. However, if no drainage is carried on this side of the building and if roof gutters are installed, the existing 5-foot-setback will be adequate and will meet the requirements of setback "b". In addition, horizontal setback requirement "d" will be met once the building is underpinned with 24-inch-deep footings.

As mentioned previously, the existing creek channel is subject to erosion and could potentially undermine the building footings. It is our understanding that the project architect and civil engineer are designing an erosion protection system for the creek that will minimize any future erosion. Plans that detail the erosion protection system will be submitted to the County when they are complete.

Allowable Bearing Capacity

A maximum allowable bearing value of 1200 pounds per square foot is recommended for new continuous footings founded at a minimum depth of 24 inches below the lowest adjacent final grade. No increase in the bearing capacity is recommended for deeper or wider footings. The above value may be increased by one-third when designing for short duration wind and seismic forces.

Lateral Resistance

A passive earth pressure of 150 pounds per square foot per foot of depth may be used to determine lateral bearing for footings. A coefficient of friction of 0.35 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. An increase of one-third



of the above values may also be used when designing for short duration wind and seismic forces. The above values are based on footings placed directly against undisturbed soil. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of maximum dry density.

Settlement

By supporting the building on 24-inch-deep footings, the upper 2 feet of porous soils will be bypassed. Based on our settlement calculations, the underpinned building may be subject to a total settlement of approximately $\frac{1}{2}$ of an inch and a differential settlement of $\frac{1}{4}$ of an inch in the future under the recommended maximum bearing value of 1200 pounds per square foot. The structural engineer should evaluate the design of the foundations to ensure that they can tolerate these settlements.

Existing Floor Slab

Based on our exploratory borings and test pit, the floor slab of building "E" is supported by colluvial materials that are subject to a moderate degree of hydroconsolidation. The slab thickness and reinforcement should be evaluated by a structural engineer to determine whether this slab is a structural slab. If the slab is a structural slab, no remediation of the underlying colluvial materials will be required. However, if the slab is not a structural slab, future settlement and cracking could occur due to the hydroconsolidation of the underlying colluvium. Although this possible cracking would be aesthetically unpleasant, it would not adversely affect the structural integrity of the building. Therefore,



at the discretion of the client, either compaction grouting may be performed in order to reduce the potential for slab settlement; the slab can be removed, the underlying unsuitable colluvial materials removed and the excavated materials replaced as properly compacted fill; or the slab may be left in its present condition and subsequently patched and repaired as future cracking occurs. Lens grouting may also be used in the future to fill in any voids that may form beneath the slab and a new layer of smooth slurry poured over the existing slab. The existing sewer and water lines should be routinely examined and tested to determine whether they have been damaged by any slab settlement.

Recommendations for Proposed Covered Bridge

Pier Foundations

The proposed bridge will be underlain by colluvial materials that are at least 11 feet deep and susceptible to a moderate to low degree of hydroconsolidation, especially within the upper 7 feet. Therefore, in order to minimize distress related to the potential adverse affects of excessive differential settlement, the footings for the covered bridge should extend through the upper colluvial materials and be founded into underlying competent materials. Due to the thickness of the unsuitable colluvial materials in this area, it is recommended that the bridge be supported on concrete pier footings extending to a minimum depth of 12 feet below grade with grade beams spanning the tops of the piers. Design recommendations are presented in the following sections.



Setbacks

By supporting the covered bridge on deepened pier footings, the required setback of Figure 2 of the County of Orange Grading Manual can be met.

Bearing Capacities

The allowable bearing capacity of each pier footing may be determined by summing the end bearing at the tip of the pier and the shaft friction around the perimeter of the pier. Allowable end bearing capacities of 3.5 and 6 kips may be used for the design of 18- and 24-inch diameter, cast-in-place concrete pier footings, respectively. The shaft friction between the pier footings and the surrounding colluvial materials may be determined by multiplying the effective overburden pressure times a coefficient of 0.19. A soil unit weight of 120 pounds per cubic foot may be used in determining the effective overburden pressure. Using this method, the shaft friction will increase as the pier depth increases; therefore, the shaft friction should be limited to a maximum value of 450 pounds per square foot.

Settlement

Under the above bearing capacities, total settlement of pier footings is expected to be less than $\frac{1}{2}$ of an inch and differential settlement is expected to be less than $\frac{1}{4}$ of an inch over a span of approximately 25 feet. The structural engineer should evaluate the design of the foundations to determine that they can tolerate these settlements.



Uplift

Pier footings may be considered to resist uplift forces equal to two-thirds of the total shaft friction value calculated using the method described above.

Lateral Resistance

A passive earth pressure of 200 pounds per square foot, per foot of depth, to a maximum value of 2000 pounds per square foot may be used to determine the lateral resistance of the pier footings.

Minimum Spacing and Depth

The project structural engineer should determine the spacing and total depths of the pier footings based on the design values presented above; however, minimum clear spacing between pier footings should be 3 pier diameters, sidewall to sidewall. Furthermore, the piers should extend to a minimum depth of 12 feet below existing grade.

Installation of Pier Footings

The colluvial materials underlying the construction site are granular with zones containing frequent to abundant gravel and occasional cobbles. Based on these conditions, some minor caving of the excavations may occur and some of the excavations may be difficult to drill. It is expected that most of the gravel and cobbles will be small enough in diameter that they can be retrieved with a bucket-auger with a wide opening, a clean-out bucket or a grab bucket. However, larger diameter cobbles may be encountered that may have to be either broken up or cored through. Rock augers, core barrels, or rock breakers may



be required to advance the excavations beyond these larger cobbles. Drilling contractors familiar with drilling granular soils that contain scattered cobbles should be consulted to determine the most economically feasible method of installing pier footings within the site.

The foundation concrete should be carefully poured into the footing excavations. If the concrete strikes the sidewalls of the excavation, it could dislodge the granular soils and cause contamination of the foundation concrete. If the concrete strikes the reinforcement cage, the concrete could segregate and become reduced in strength. For these reasons, a tremie pipe should be used to properly place the foundation concrete.

Geotechnical Observations

All excavations for pier footings should be observed by the project geotechnical consultant to verify that they are cast against the anticipated geotechnical conditions, that the pier excavations are properly prepared, and that the proper dimensions are achieved.

Grade Beams

Due to the potential for future settlement of the colluvial materials, the grade beams should be designed as structural members that do not derive any support from the underlying soils.

Bridge Abutment

Retaining walls associated with construction of the bridge abutment should be designed in accordance with the recommendations previously presented for the easterly wall of Building A-D. Footings for the retaining walls should be ex-



tended through the porous and compressible surface soils and founded in underlying competent bearing materials. Due to the thickness of the unsuitable colluvial materials, it may be necessary to support the retaining walls on pier footings and grade beams.

Recommendations for Proposed Entrance Gate

Site Grading

The area of the proposed entrance gate is underlain by colluvial materials that are at least 11 feet deep and are subject to a moderate degree of hydroconsolidation within the upper 3 feet. Therefore, in order to mitigate distress related to the potential adverse affects of excessive total and differential settlements, the existing ground surface to a depth of 3 feet within the footing areas should be removed and recompactd. Recommendations for this remedial grading are presented in the "Earthwork" section of this report.

Allowable Bearing Capacity and Lateral Resistance

If remedial grading is performed as described above, the allowable bearing values and lateral resistance values recommended for design of the footings for Building A-D may be used for design of pad footings supporting the entrance gate pilasters.

Earthwork

General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with the following recommendations prepared by this firm, and all applicable requirements of the Grading Manual of the County of Orange, California.



Site Clearing

Prior to any grading, all significant vegetation existing within areas to be graded should be stripped and removed from the site. Should any buried structures or unusual soil conditions be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Processing of Existing Ground

1. Building A-D

Surface soils to a depth of 3 feet within the area of Building A-D are subject to a moderate degree of hydroconsolidation. Therefore, in order to mitigate distress related to the potential adverse affects of excessive total and differential settlements, the existing ground surface to a depth of 3 feet should be recompact to a minimum relative compaction of 90 percent. This should be accomplished by overexcavating the existing ground surface to a depth of 2.5 feet and then processing the next underlying 6 inches in place. That is, prior to replacing the overexcavated soils as properly compacted fill, the exposed bottom surface should first be scarified to a depth of 6 inches, watered or dried as necessary to achieve near optimum moisture conditions, and recompact in place to a minimum relative density of 90 percent. Horizontal limits of overexcavation and recompact-ion should extend to a distance of at least 3 feet beyond the perimeter footings (see Cross Section C-C', Plate 2).



2. Entrance Gate

Surface soils to a depth of 3 feet within the area of the proposed entrance gate are also subject to a moderate degree of hydroconsolidation. Therefore, in order to mitigate distress related to the potential adverse affects of excessive total and differential settlements, the existing ground surface to a depth of 3 feet within the footing areas should be recompacted to a minimum relative compaction of 90 percent. This should be accomplished by overexcavating the existing ground surface to a depth of 2.5 feet and then processing the next underlying 6 inches in place. That is, prior to replacing the overexcavated soils as properly compacted fill, the exposed bottom surface should first be scarified to a depth of 6 inches, watered or dried as necessary to achieve near optimum moisture conditions, and re-compacted in place to a minimum relative density of 90 percent. Horizontal limits of overexcavation and recompaction should extend to a distance of at least 3 feet beyond the footings.

Fill Placement and Testing

All fills should be placed in lifts not exceeding 6 inches in thickness, watered or air-dried as necessary to achieve a moisture content that is 2 to 5 percent above optimum moisture content, thoroughly blended, then compacted in place to a minimum relative compaction of 90 percent. Each fill lift should be treated in a similar manner. Subsequent lifts should not be placed until the preceding lift has been tested and approved by the project geotechnical consultant.

The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557-91.



Geotechnical Observations

Exposed bottom surfaces in each removal area should be observed and approved by the project geotechnical consultant prior to placing fill. No fills should be placed without prior approval from the geotechnical consultant.

A representative of the geotechnical consultant should be present onsite during grading operations to verify proper placement and compaction of all fills, as well as to verify compliance with the other recommendations presented herein.

Shrinkage and Subsidence

An average shrinkage factor estimated at approximately 15 percent will occur when excavated onsite soils are replaced as properly compacted fill. A subsidence estimated at approximately 0.10 feet will also occur when exposed bottom surfaces in removal areas are scarified and recompactd as recommended herein.

The above estimates are intended as an aid for project planners in determining earthwork quantities. However, these estimates should be used with caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities on the basis of actual shrinkage and subsidence that will occur during grading.

Post-Grading Considerations

Site Drainage

Positive drainage facilities such as sloping concrete flatwork and graded earth swales should be provided around the new construction to collect and direct all surface waters away from structure foundations and building walls.



Utility Trench Backfill

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Where onsite soils are utilized as backfill, mechanical compaction will be required. Density testing, along with probing, should be performed by a representative of the project geotechnical consultant to verify adequate compaction.

Backfill should be placed in approximately 12- to 18-inch thick maximum lifts, and then mechanically compacted with a hydra-hammer, pneumatic tamper, or similar equipment that can achieve a minimum relative compaction of 90 percent.

As an alternate for shallow trenches where pipe may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand having a Sand Equivalent value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to verify adequate compaction.

Where utility trenches are proposed parallel to any building footing (interior or exterior trench), the bottom of the trench should not extend below a 1:1 plane projected downward from the outside bottom edge of the adjacent footing. Where this condition occurs, the adjacent footing should be deepened, or the trench backfilled with sand-cement slurry.



Soluble Sulfate Analysis

Laboratory test results indicate that onsite soils contain less than 0.10 percent water soluble sulfate contents. According to Table 26-A-3 of the 1991 Uniform Building Code, no special sulfate resistance cement will be necessary for concrete placed in contact with the onsite soils.

Seismicity

The maximum credible earthquake for a particular fault is the largest magnitude event that can reasonably be postulated to occur based upon existing geologic and seismologic evidence independent of time. Most maximum credible earthquakes generally cannot be assigned a meaningful probability of occurrence, which is usually very low over the useful design life of most construction. The Elsinore fault zone would probably generate the most severe maximum credible site ground motions with a maximum credible magnitude of 7.0 on the Richter scale.

Estimated peak site ground acceleration from a magnitude 7.0 earthquake on the Elsinore Fault, should an event occur opposite the site, is on the order of 0.22g (Joyner and Fumal, 1985). Repeatable ground accelerations are generally considered to be equal to two-thirds of the maximum peak accelerations; thus $67\% \times .22g = 0.15g$.

The property will probably experience ground shaking from at least small to moderate size earthquakes during the life of the proposed structure. Furthermore, it should be recognized that the Southern California region is an area of moderate to high seismic risk and that it is not considered feasible to make structures totally resistant to seismic related hazards. The accelerations pre-



viously mentioned are presented for consideration; however, the design acceleration should be determined by the structural consultant, and be reflective of the type of structure proposed. Design in accordance with the current Uniform Building Code and the seismic design parameters of the Structural Engineers Association of California is expected to satisfactorily mitigate the effects of ground shaking.

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure as well as induced flooding. Various general types of ground failures which might occur as a consequence of severe ground shaking of the site include landsliding, ground subsidence, ground lurching, shallow ground rupture, and liquefaction. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors.

Based on the nature of soil and groundwater conditions beneath the property, all of the above secondary effects of seismic activity are considered unlikely at the site.

CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of **MR. AND MRS. RICK LAWRENCE** to assist the Project Engineer and Architect for site and building evaluation. It is recommended that we be engaged to review any revised foundation plans in order to verify that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. If we are not accorded the opportunity to review these documents, we



take no responsibility for misinterpretation of our recommendations. Depending on the results of this review, additional recommendations and/or modifications may be necessary.

We recommend that we be retained to provide soil engineering services during construction of the excavation and foundation phases of the work. This is to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. If conditions are encountered during construction that appear to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

INVESTIGATION LIMITATIONS

This report is based on the project as described and the geotechnical data obtained from the field tests performed at the location indicated on the plan. The materials encountered on the project site and utilized in our laboratory investigation are believed representative of the total area. However, soils can vary in characteristics, both laterally and vertically. The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our best professional judgement. The findings, conclusions, and opinions contained in this report are to be considered tentative only, and are subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and this firm or the undersigned professionals assume no responsibility for its use.



MR. AND MRS. RICK LAWRENCE

November 9, 1995

J.N. 147-95

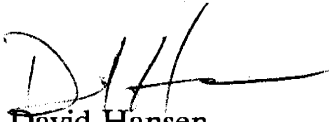
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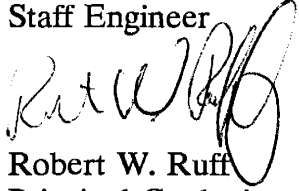
This report has not been prepared for use by parties or projects other than those named or described above. It may not contain sufficient information for other parties or other purposes.

This opportunity to be of service is sincerely appreciated. Please call if you have any questions pertaining to this report.

Respectfully submitted,

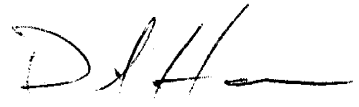
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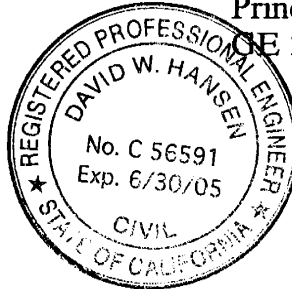

David Hansen
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- 2) Response to County Review Sheet dated December 14, 1989, by Action Geotechnical, dated January 11, 1990, W.O. 557701-A.
- 3) Change in Geotechnical Consultants, 19191 Lawrence Canyon Rd., by Strata-Tech, Inc., dated August 7, 1990.
- 4) Geotechnical Investigation of Phase I, Rancho Los Lomas Property, 19191 Lawrence Canyon Rd., County of Orange, by Strata-Tech, Inc., dated August 1, 1990, W.O. 27190.
- 5) Geotechnical Investigation Proposed Residential Sites, 19191 Lawrence Canyon Rd., County of Orange, by Strata-Tech, Inc., dated September 18, 1990, W.O. 30390.
- 6) Response to County of Orange Review Sheet dated September 20, 1990, for 19191 Lawrence Canyon Rd., OCPC# 648-89G, by Strata-Tech, Inc., dated March 26, 1991, W.O. 27190-01.
- 7) Response to County of Orange Review Sheet dated May 8, 1991, Residential Sites, GPC 640-89G, by Strata-Tech, Inc., dated October 4, 1991, W.O. 30390-A.
- 8) Response to County of Orange Review Sheet dated April 19, 1991, Phase I Investigation, GPC# 640-89G, by Strata-Tech, Inc., dated October 8, 1991, W.O. 27190-02.
- 9) Digest of Geotechnical Conditions and Recommendations, 19191 Lawrence Canyon Rd., by Strata-Tech, Inc., dated January 15, 1992, W.O. 27190-03.
- 10) Geotechnical Investigation of Phase II, Rancho Los Lomas, 19191 Lawrence Canyon Rd., County of Orange, by Strata-Tech, Inc., dated January 21, 1992, W.O. 27190-04.
- 11) Response to County of Orange Review Sheet dated February 27, 1992, by Strata-Tech, Inc., dated April 22, 1992, W.O. 27190.
- 12) Response to County of Orange Review Sheet dated May 8, 1992, by Strata-Tech, Inc., dated July 15, 1992, W.O. 27190-05.
- 13) Variance Package, Oversteepened Slopes, 19191 Lawrence Canyon Rd., by Strata-Tech, Inc., dated July 22, 1992.
- 14) Geotechnical Data for Variance Request, Parking Area, by Strata-Tech, Inc., dated August 5, 1992, W.O. 27190-06-1.

PETRA GEOTECHNICAL, INC.
J.N. 147-95



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(cont'd.)

- 15) Geotechnical Data for Variance Request, Berm Area, by Strata-Tech, Inc., dated August 5, 1992, W.O. 27190-06-2.
- 16) Geotechnical Data for Variance Request, Dam Areas, by Strata-Tech, Inc., dated August 5, 1992, W.O. 27190-06-3.
- 17) Response to County of Orange Review Sheet dated August 27, 1992, for Variance of Oversteepened Slope (S1-S18) by Strata-Tech, Inc., dated September 7, 1992, W.O. 27190-07.
- 18) Response to County of Orange Review Sheet dated August 12, 1992, by Strata-Tech, Inc., dated September 7, 1992, W.O. 27190-08.
- 19) Geotechnical Data for Setback Variance Request for Buildings A, B, D, E, G, R, U, X and Y, 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated September 23, 1992, W.O. 27190-09.
- 20) Geotechnical Investigation of Existing Pool and Descending Slope, 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated October 26, 1992, W.O. 27190-10.
- 21) Limited Investigation of the Hardscape at 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated October 28, 1992, W.O. 27190-11.
- 22) Response to Geotechnical Review Sheet, dated September 30, 1992 for 19191 Lawrence Canyon Rd., County of Orange, California by Strata-Tech, Inc., dated October 28, 1992, W.O. 27190-12.
- 23) Geotechnical Investigation of Buildings H, A-A, and A-B (Phase III), Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated February 27, 1995.
- 24) Geotechnical Investigation, Proposed Greenhouse, Tool Storage Room, Restrooms, and Existing Building "C", Rancho Las Lomas, 19191 Lawrence Canyon Road, County of Orange, California; report by Petra Geotechnical, Inc., dated October 20, 1995.

PETRA GEOTECHNICAL, INC.
J.N. 147-95



APPENDIX A

EXPLORATION LOGS

TEST PIT LOG



EXPLORATION LOG

Project: Rancho Las Lomas						Boring No.: B-12					
Location: 19191 Lawrence Canyon Road, County of Orange						Elevation:					
Job No.: 147-95			Client: Lawrence			Date: 9/1/95					
Drill Method: Hand Driven			Driving Weight: Hand Augered			Logged By: D. Hansen					
Depth (Feet)	Lithology	Material Description	Water	Samples		Laboratory Tests					
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests		
		COLLUVIUM									
		Clayey Sand (SC): Brown; moist; medium dense to occasionally dense; slightly porous to porous in local zones; fine to medium grained; scattered gravel and cobbles.					7.3	101.8	MAX EXP SO4 CON		
5		Silty Sand (SM): Yellow brown; moist; medium dense; slightly porous to porous in local zones; fine to medium grained; abundant gravel. Abundant gravel and angular cobbles, decrease in porosity, increase in density.					8.2	104.7	DSU		
10							8.6	111.4	CON		
Total depth 11 feet. Slight caving. No groundwater. Other laboratory tests: MAX - Maximum Density. EXP - Expansion Potential. SO4 - Soluble Sulfate Content. DSU - Direct Shear, Undisturbed Sample. CON - Consolidation Test.											



EXPLORATION LOG

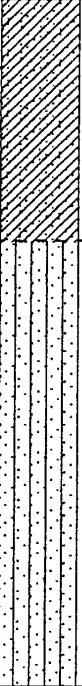
Project: Rancho Las Lomas				Boring No.: B-13						
Location: 19191 Lawrence Canyon Road, County of Orange				Elevation:						
Job No.: 147-95		Client: Lawrence		Date: 9/1/95						
Drill Method: Hand Driven		Driving Weight: Hand Augered		Logged By: D. Hansen						
Depth (Feet)	Lith- ology	Material Description	W a t e r	Samples		Laboratory Tests				
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests	
		COLLUVIUM <u>Clayey Sand (SC)</u> : Brown; moist; medium dense to occasionally dense; slightly porous to porous in local zones; fine to medium grained; scattered gravel and cobbles.								
5		<u>Silty Sand (SM)</u> : Yellow brown; moist; medium dense; slightly porous to porous in local zones, fine to medium grained; abundant gravel.								
		Abundant gravel and angular cobbles, decrease in porosity, increase in density.								
					</					

PLATE A-2





EXPLORATION LOG

Project: Rancho Las Lomas				Boring No.: B-14			
Location: 19191 Lawrence Canyon Road, County of Orange				Elevation:			
Job No.: 147-95		Client: Lawrence		Date: 9/1/95			
Drill Method: Hand Driven		Driving Weight: Hand Augered		Logged By: D. Hansen			

Depth (Feet)	Lith- ology	Material Description	W a t e r	Samples			Laboratory Tests		
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		<u>COLLUVIUM</u> <u>Clayey Sand (SC)</u> : Brown; moist; medium dense to occasionally dense; slightly porous to porous in local zones; fine to medium grained; scattered gravel and cobbles. <u>Silty Sand (SM)</u> : Yellow brown; moist; medium dense; slightly porous to porous in local zones, fine to medium grained; abundant gravel. Abundant gravel and angular cobbles, decrease in porosity, increase in density.							
5									
10									
		Total depth 10 feet, refusal due to gravel and cobbles. Slight caving. No groundwater. <u>Other laboratory tests:</u> CON - Consolidation Test.							



EXPLORATION LOG

Project: Rancho Las Lomas				Boring No.: B-15					
Location: 19191 Lawrence Canyon Road, County of Orange				Elevation:					
Job No.: 147-95		Client: Lawrence		Date: 9/1/95					
Drill Method: Hand Driven		Driving Weight: Hand Augered		Logged By: D. Hansen					
Depth (Feet)	Lith- ology	Material Description	W a t e r	Samples			Laboratory Tests		
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
5		COLLUVIUM <u>Clayey Sand (SC)</u> : Brown; moist; medium dense and slightly porous to porous in upper 4 feet, dense below; fine to medium grained; scattered gravel and cobbles.					10.2	102.5	CON
		<u>Silty Sand (SM)</u> : Yellow brown; moist; dense; fine to medium grained; abundant gravel and frequent angular cobbles.					9.8	113.1	CON
							7.5	116.3	
Total depth 9 feet, refusul due to gravel and cobbles. Slight caving. No groundwater. <u>Other laboratory tests:</u> CON - Consolidation Test.									



EXPLORATION LOG

Project: Rancho Las Lomas				Boring No.: B-16						
Location: 19191 Lawrence Canyon Road, County of Orange				Elevation:						
Job No.: 147-95		Client: Lawrence		Date: 9/2/95						
Drill Method: Hand Driven		Driving Weight: Hand Augered		Logged By: D. Hansen						
Depth (Feet)	Lith- ology	Material Description	W a t e r	Samples		Laboratory Tests				
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests	
5		<u>COLLUVIUM</u> <u>Sandy Silt (ML)</u> : Yellow brown; soft to firm and porous in upper 3 feet, firm to stiff below; occasional layers with fine grained sand.					12.7	102.3	MAX EXP SO4 DSR CON	
								15.2	108.7	CON
10		<u>Silty Sand (SM)</u> : Yellow brown; moist; dense; fine grained; trace gravel.				11.5	110.7			
			</							

PLATE A-5



EXPLORATION LOG

Project: Rancho Las Lomas				Boring No.: B-17			
Location: 19191 Lawrence Canyon Road, County of Orange				Elevation:			
Job No.: 147-95		Client: Lawrence		Date: 9/2/95			
Drill Method: Hand Driven		Driving Weight: Hand Augered		Logged By: D. Hansen			

Depth (Feet)	Lith- ology	Material Description	W a t e r	Samples			Laboratory Tests		
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		<u>COLLUVIUM</u>							
		<u>Sandy Silt (ML)</u> : Yellow brown; soft to firm and porous in upper 3 feet, firm to stiff below; occasional layers with fine grained sand.					11.8	100.6	
5		Sand layers more frequent.					12.2	106.4	
		Scattered gravel, frequent sand layers.							
10		Total depth 10 feet. No caving. No groundwater.							

PLATE A-6



EXPLORATION LOG



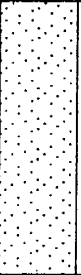
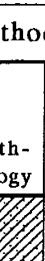

Project: Rancho Las Lomas				Boring No.: B-18					
Location: 19191 Lawrence Canyon Road, County of Orange				Elevation:					
Job No.: 147-95		Client: Lawrence		Date: 9/2/95					
Drill Method: Hand Driven		Driving Weight: Hand Augered		Logged By: D. Hansen					
Depth (Feet)	Lith- ology	Material Description	W a t e r	Samples			Laboratory Tests		
				Blows Per Foot	C o r e	B u l k	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
5		COLLUVIUM <u>Clayey Sand (SC):</u> Brown; dry to slightly moist; medium dense and porous in upper 2 to 3 feet, dense below; fine grained; some roots. Increase in density and decrease in porosity.					13.5	93.9	MAX EXP SO4 DSR CON
10		<u>Silty Sand (SM):</u> Yellow brown; slightly moist; dense; fine to medium grained; frequent gravel. Frequent gravel.					10.0	106.9	CON
15		BEDROCK - Sespe Formation <u>Sandstone:</u> Yellow brown; moist; dense; fine to medium grained; weathered.					7.8	114.3	
		Total depth 15 feet. No caving. No groundwater. Other laboratory tests: MAX - Maximum Density. EXP - Expansion Potential. SO4 - Soluble Sulfate Content. DSR - Direct Shear, Remolded Sample. CON - Consolidation Test.							

PLATE A-7



EXPLORATION LOG

Project: Rancho Las Lomas						Boring No.: B-19					
Location: 19191 Lawrence Canyon Road, County of Orange						Elevation:					
Job No.: 147-95			Client: Lawrence			Date: 9/2/95					
Drill Method: Hand Driven			Driving Weight: Hand Augered			Logged By: D. Hansen					
Depth (Feet)	Lithology	Material Description	Water	Samples		Laboratory Tests					
				Blows Per Foot	Cores	Bulk	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests		
5		COLLUVIUM									
		Clayey Sand (SC): Brown; dry to slightly moist; medium dense and porous in upper 3 feet, dense below; fine grained; some roots.									
		Increase in density and decrease in porosity.									
		Silty Sand (SM): Yellow brown; slightly moist; dense; fine to medium grained; frequent gravel.									
		Total depth 9 feet. No caving. No groundwater.									



TEST PIT LOG

Project Number: J.N. 147-95

Logged By: D. Hansen

Test Pit No.: TP-10

Project Name: Lawrence

Date: Sept. 2, 1995

Elevation: _____

Equipment: Hand-Excavated

Remarks: _____

DEPTH	SOIL/BEDROCK DESCRIPTION
0.0 - 3.0	<p>COLLUVIUM (Qcol)</p> <p>Clayey Sand (SC): Brown, slightly moist, medium dense, porous, fine to medium grained, scattered gravel, increase in sand with depth.</p> <p>Footing only 6 inches deep, may be just a thick slab on grade.</p> <p>Total Depth - 3 Feet No Groundwater No Caving</p>

SCALE: 1" = 5'

WALL SHOWN:

TREND:

NO.	STRUCTURAL ATTITUDES

PETRA GEOTECHNICAL, INC.

TEST PIT NO. TP-10

PROJECT NO. 147-95

APPENDIX B

LABORATORY TEST CRITERIA

LABORATORY TEST DATA



LABORATORY TEST CRITERIA

Soil Classification

Soils encountered within the property were classified and described utilizing the visual-manual procedures of the Unified Soil Classification System, and in general accordance with Test Method ASTM D 2488-84. The assigned group symbols are presented in the "Test Pit Log" and in the "Exploration Logs," Appendix A.

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content of onsite soils were determined for selected samples in accordance with Method A of ASTM D 1557-91. Pertinent test values are given on Plate B-1.

Expansion Potential

Expansion index tests were performed on selected samples of soil in accordance with Uniform Building Code Test No. 29-2. The results of these tests are presented on Plate B-1.

Soluble Sulfate Analysis

Soluble sulfate analyses were performed on selected samples of soil to determine water soluble sulfate content. These tests were performed in accordance with California Test Method No. 417. Test results are included on Plate B-1.

In Situ Moisture and Density

Moisture content and unit dry density of the in place soils were determined in representative strata. Test data are presented in the "Exploration Logs," Appendix A.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for samples of colluvial materials remolded to 90 percent of maximum density and for an undisturbed sample of colluvium. These tests were performed in general accordance with Test Method No. ASTM D-3080. Three test specimens were prepared for each test, artificially saturated, then sheared under varying normal loads at a constant rate of strain of 0.05 inches per minute. Results are graphically presented on Plates B-2 and B-3.

Consolidation

Consolidation tests were performed in general accordance with Test Method ASTM D 2435-80. Axial loads were applied in several increments to a laterally restrained 1-inch high sample. Loads were applied in a geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were inundated at a surcharge loading approximately equal to the existing or proposed total overburden pressures in order to evaluate the effects of a sudden increase in moisture content (hydroconsolidation potential). Results of these tests are graphically presented on Plates B-4 through B-8.



LABORATORY MAXIMUM DRY DENSITY*

<u>Boring Number</u>	<u>Depth (ft.)</u>	<u>Soil Type</u>	<u>Optimum Moisture (%)</u>	<u>Maximum Dry Density (pcf)</u>
B-12	2	A - Clayey Sand	11.0	123.0
B-16	1	B - Sandy Silt	12.0	121.0
B-18	1	C - Clayey Sand	10.0	125.0

EXPANSION INDEX TEST DATA**

<u>Soil Type</u>	<u>Expansion Index</u>	<u>Expansion Potential***</u>
A - Clayey Sand	48	Low
B - Sandy Silt	54	Medium
C - Clayey Sand	42	Low

SOLUBLE SULFATES****

<u>Soil Type</u>	<u>Sulfate Content (%)</u>
A - Clayey Sand	.0232
B - Sandy Silt	.0975
C - Clayey Sand	.0150

* Per Test Method ASTM D1557-91.

** Per Uniform Building Code Standard Test No. 29-2.

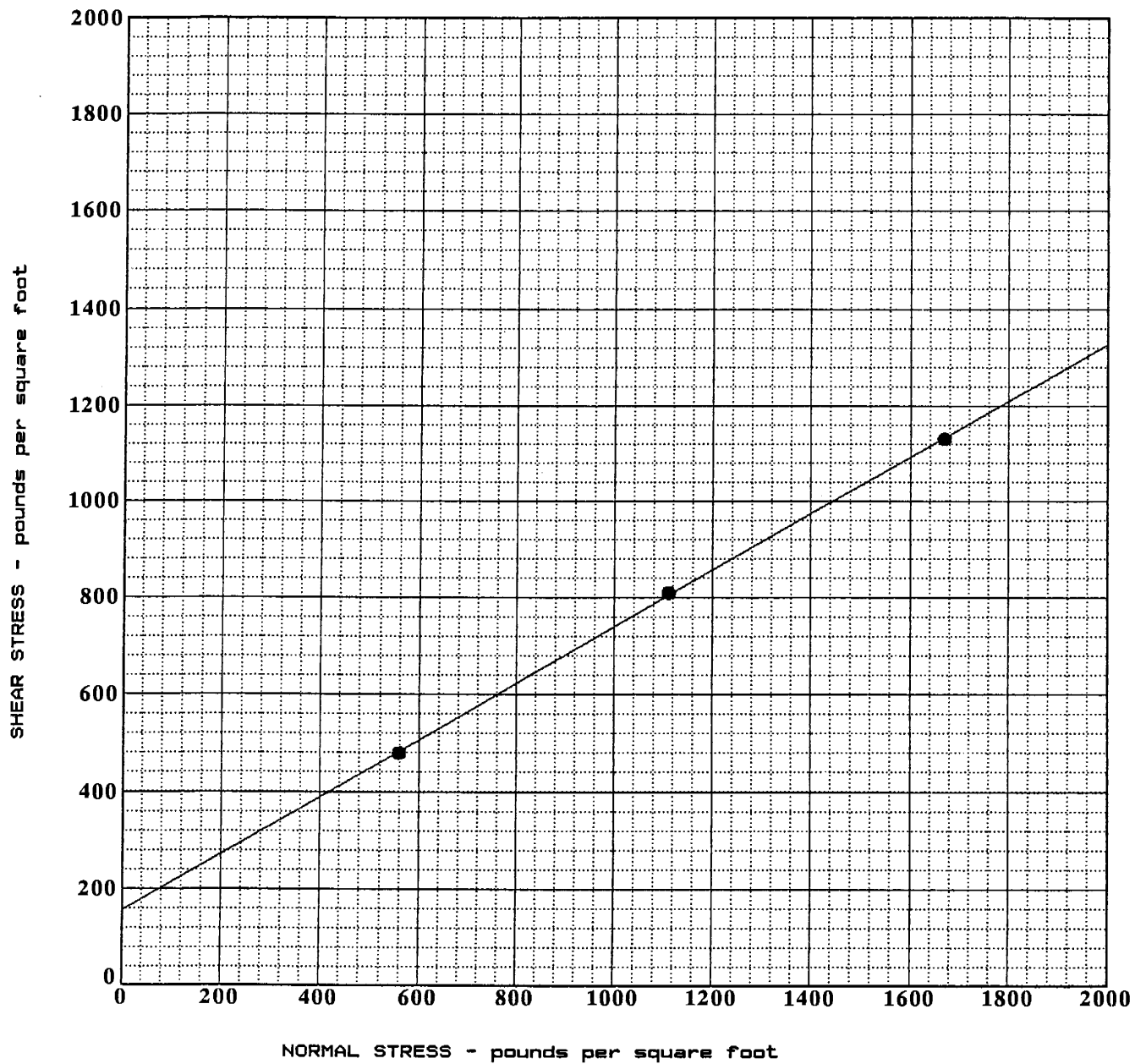
*** Per Table 29-C, "Classification of Expansive Soils",
Uniform Building Code, latest edition.

**** Per California Test Method No. 417.

PLATE B-1

PETRA GEOTECHNICAL, INC.





SAMPLE LOCATION	DESCRIPTION	FRICTION ANGLE	COHESION (PSF)
● B-12 @ 5.0	Silty Sand (SM)	30	155

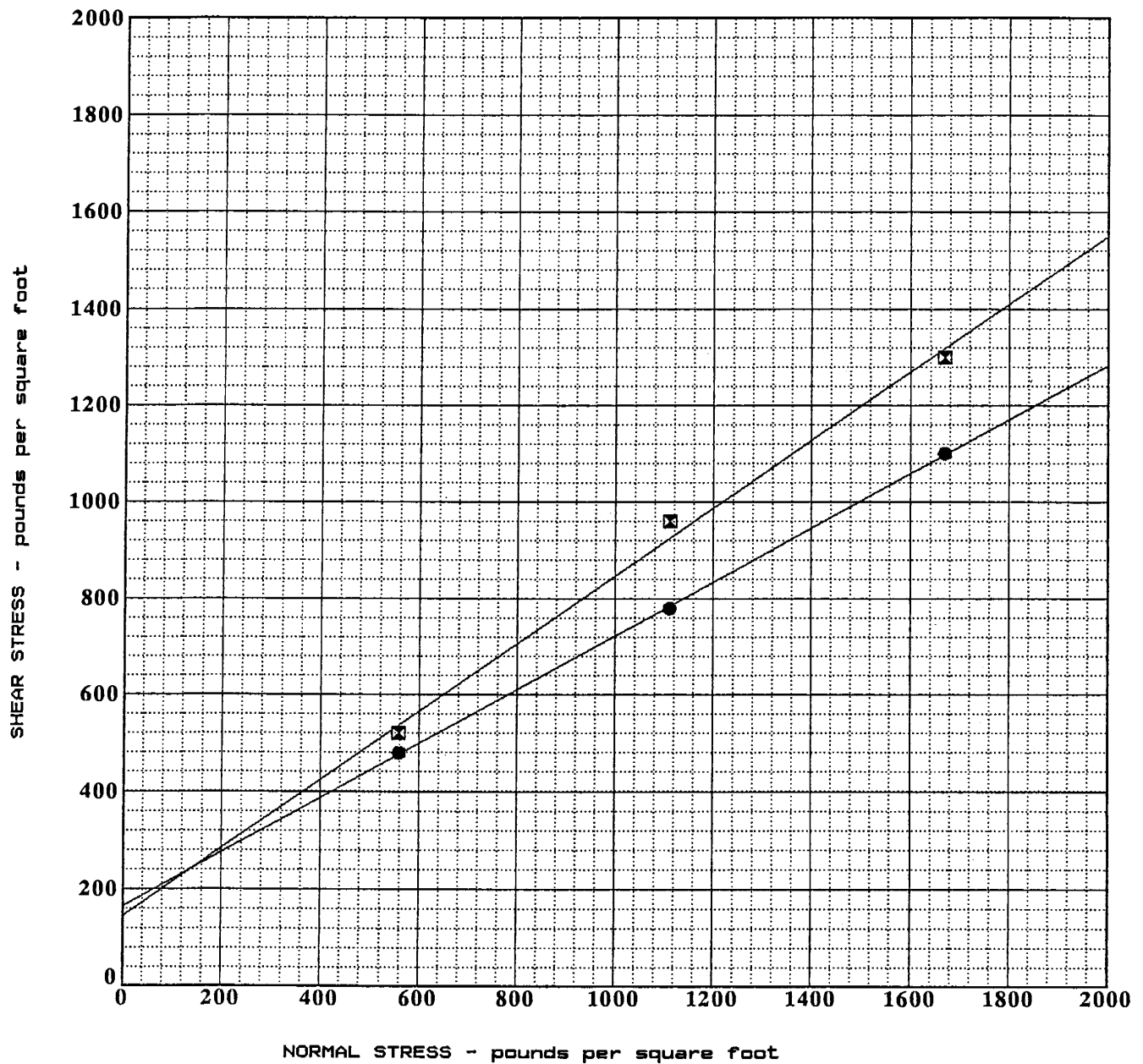
NOTES:

Undisturbed Test Samples

All Samples Were Inundated Prior to Shearing

J.N. 147-95	DIRECT SHEAR TEST DATA UNDISTURBED TEST SAMPLES	November, 1995
PETRA GEOTECHNICAL, INC.		PLATE B-7





SAMPLE LOCATION	DESCRIPTION	FRICTION ANGLE	COHESION (PSF)
● B-16 @ 2.0	Sandy Silt (ML)	29	165
⊠ B-18 @ 2.0	Clayey Sand (SC)	35	145

NOTES:

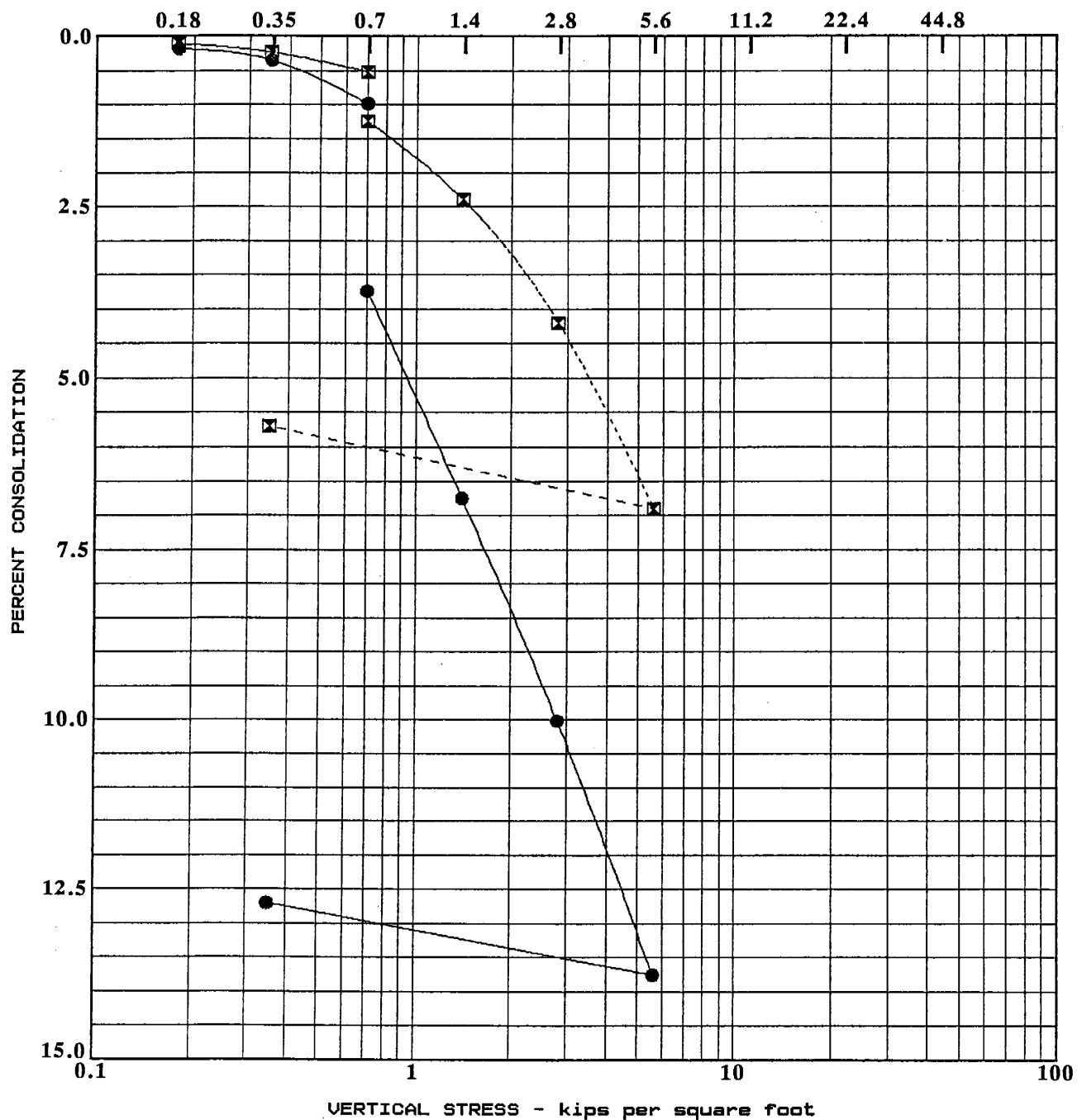
Samples Remolded to 90 % of Maximum Dry Density

All Samples Were Inundated Prior to Shearing

J.N. 147-95	DIRECT SHEAR TEST DATA REMOLDED TEST SAMPLES	November, 1995
PETRA GEOTECHNICAL, INC.		PLATE B-3



SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-12 @ 2.0	Clayey Sand (SC)	101.8	7.3	30	0.70
☒ B-12 @ 8.0	Silty Sand (SM)	111.4	8.6	45	0.70



J.N. 147-95

PETRA GEOTECHNICAL, INC.

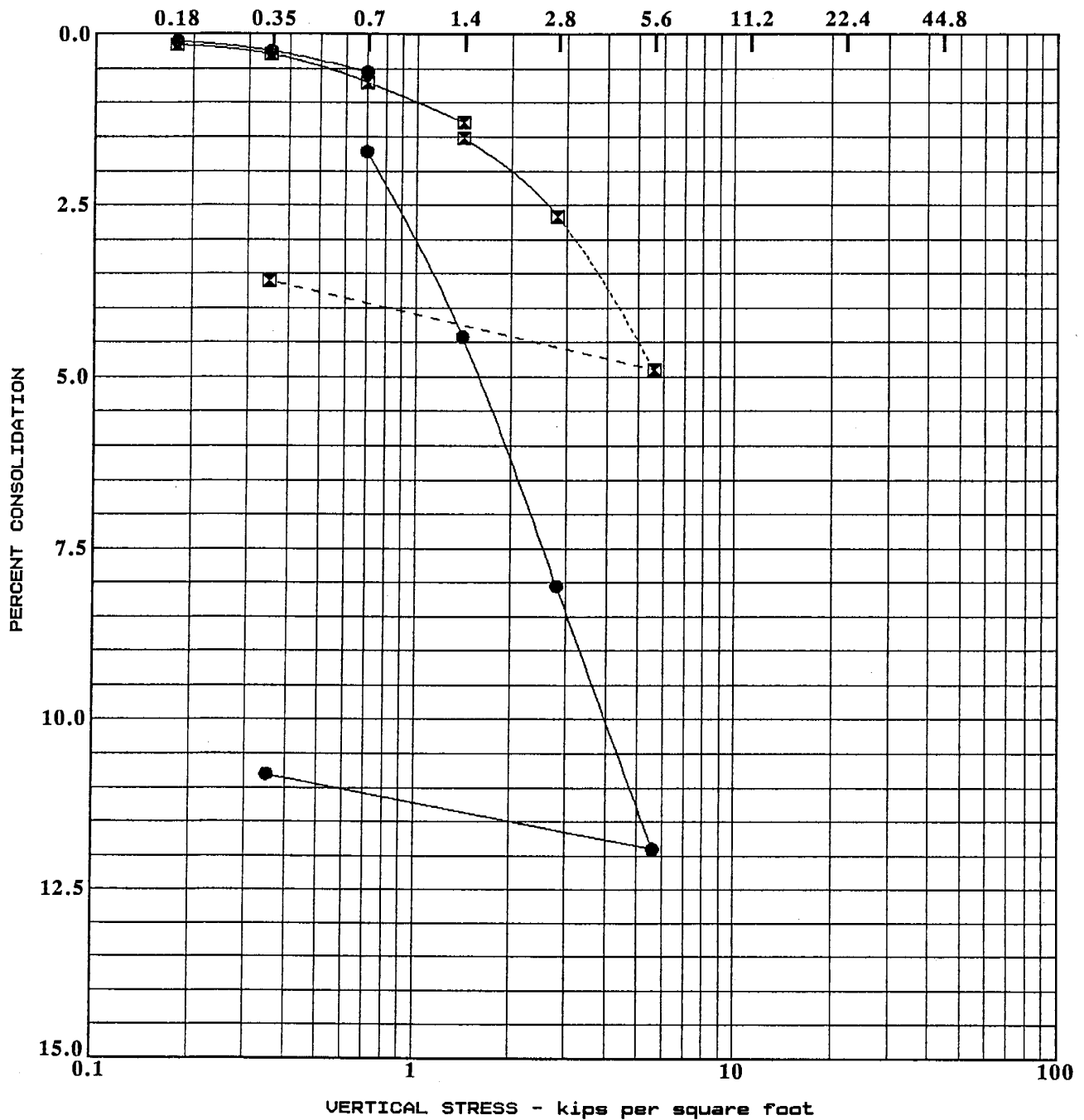
CONSOLIDATION TEST RESULTS

November, 1995

PLATE B-4



SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-13 @ 3.0	Clayey Sand (SC)	98.9	8.3	32	0.70
☒ B-13 @ 9.0	Silty Sand (SM)	113.2	9.9	55	1.40



J.N. 147-95

PETRA GEOTECHNICAL, INC.

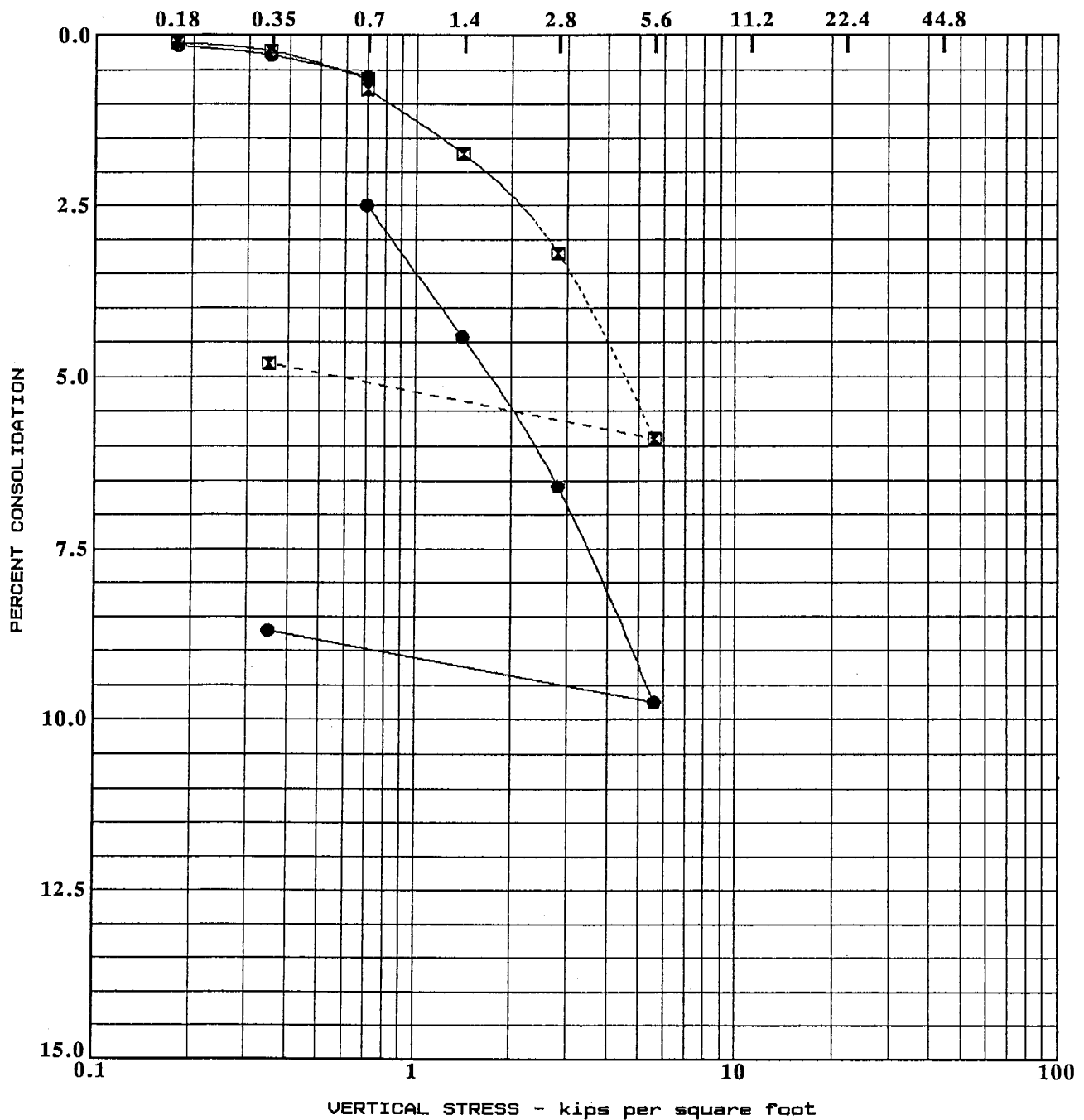
CONSOLIDATION TEST RESULTS

November, 1995

PLATE B-5



SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-15 @ 2.0	Clayey Sand (SC)	102.5	10.2	43	0.70
☒ B-15 @ 5.0	Clayey Sand (SC)	113.1	9.8	54	0.70



J.N. 147-95

PETRA GEOTECHNICAL, INC.

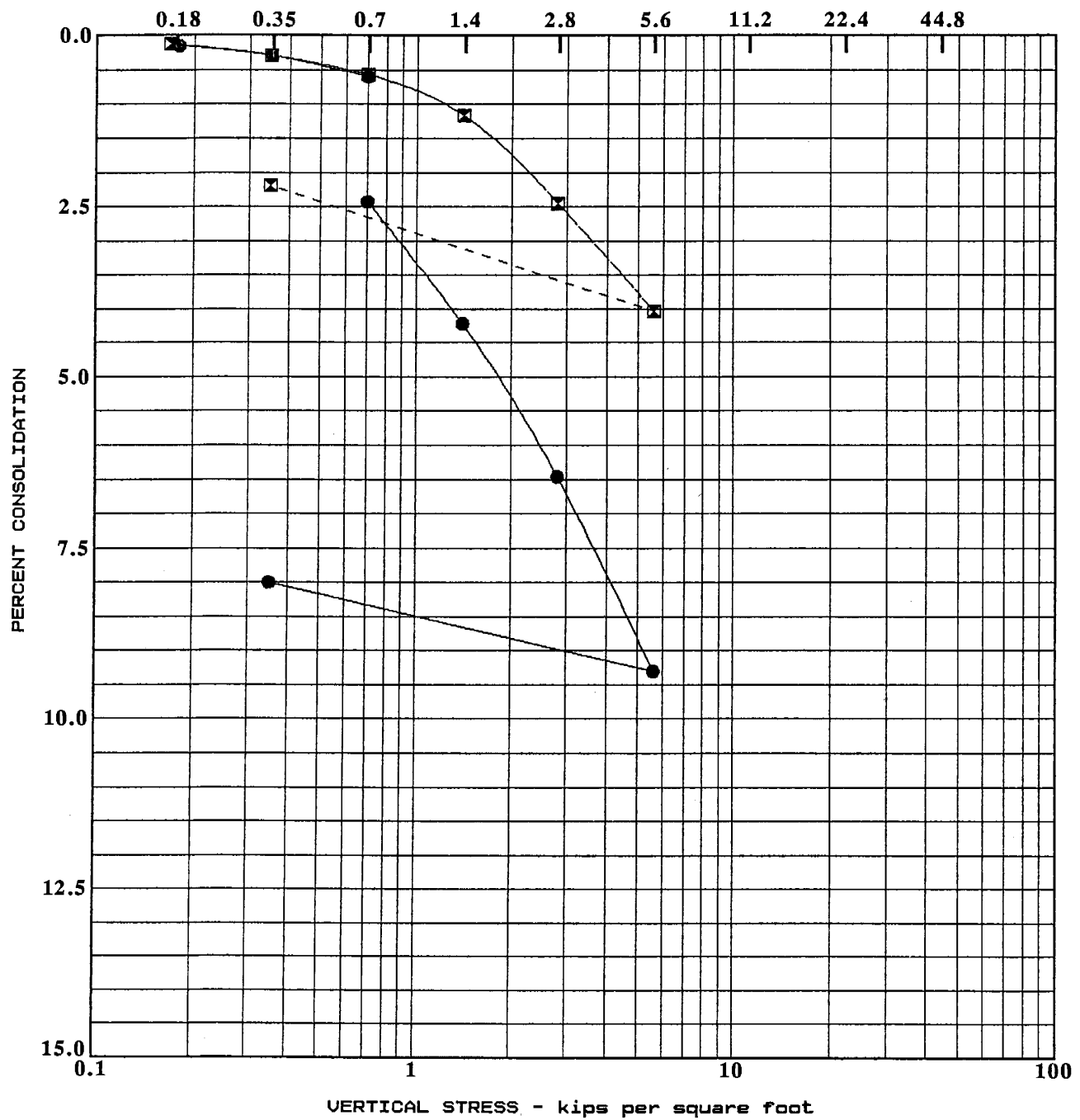
CONSOLIDATION TEST RESULTS

November, 1995

PLATE B-6



SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-16 @ 2.0	Sandy Silt (ML)	102.3	12.7	53	0.70
☒ B-16 @ 6.0	Sandy Silt (ML)	108.7	15.2	75	0.70



J.N. 147-95

PETRA GEOTECHNICAL, INC.

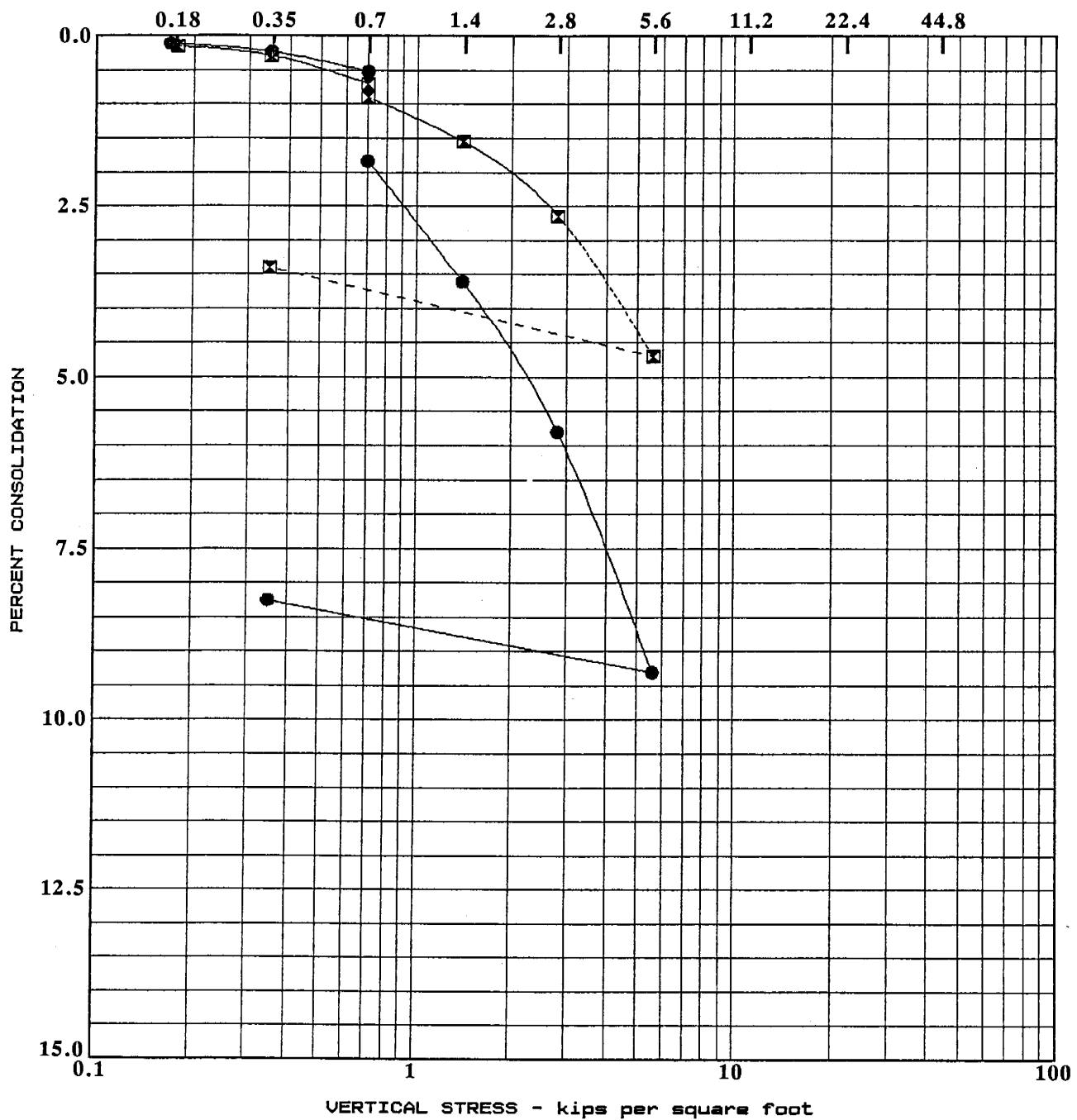
CONSOLIDATION TEST RESULTS

November, 1995

PLATE B-7



SAMPLE LOCATION	MATERIAL DESCRIPTION	INITIAL			INUNDATED
		DENSITY (pcf)	MOISTURE (%)	SATURATION (%)	LOAD (ksf)
● B-18 @ 2.5	Clayey Sand (SC)	93.9	13.5	46	0.70
☒ B-18 @ 6.0	Clayey Sand (SC)	106.9	10.0	47	0.70



J.N. 147-95

PETRA GEOTECHNICAL, INC.

CONSOLIDATION TEST RESULTS

November, 1995

PLATE B-8



APPENDIX C

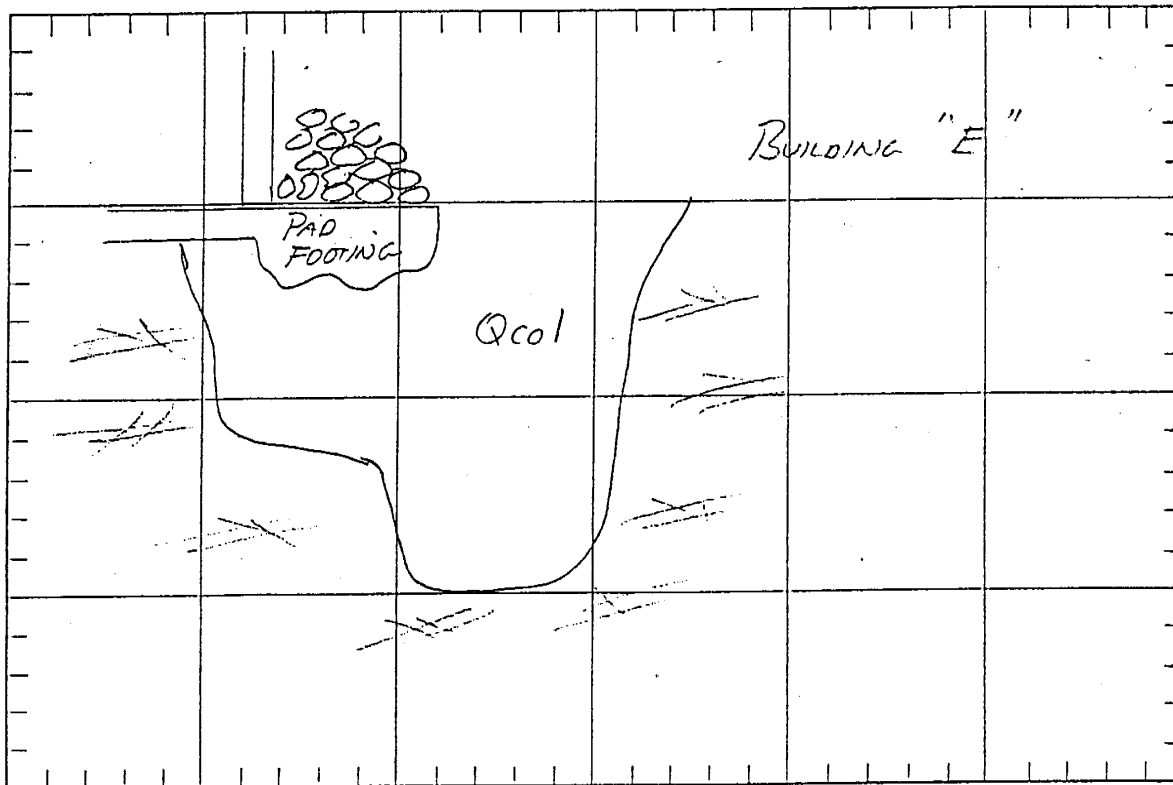
**LOG OF PREVIOUS TEST PITS
(STRATA-TECH)**



RECORD OF SUBSURFACE EXPLORATION

W.O. # 27190				Project: Rick Lawrence		Date: 12-13-91		TEST PIT # 8	
USCS	Samples	Moisture %	Dry Density (pcf)	Depth (ft.)	Logged by: D.H.		Elevation:		
					Equipment: Hand		Samples: B: Bag R: Ring T: Tube		
					Description of Earth Materials				
CL	R	9	102	1	Dark Brown Sandy Clay, Moist to Slightly Moist, Soft to Firm with Depth (Qcol)				
				2					
ML	R	9	114	3	Red Brown Clayey, Sandy, Silt, Moist, Firm to Stiff (Qcol)				
				4					
					EOB @ 4' No Water, No Caving				

CROSS SECTION SKETCH



Scale 1" = 2'

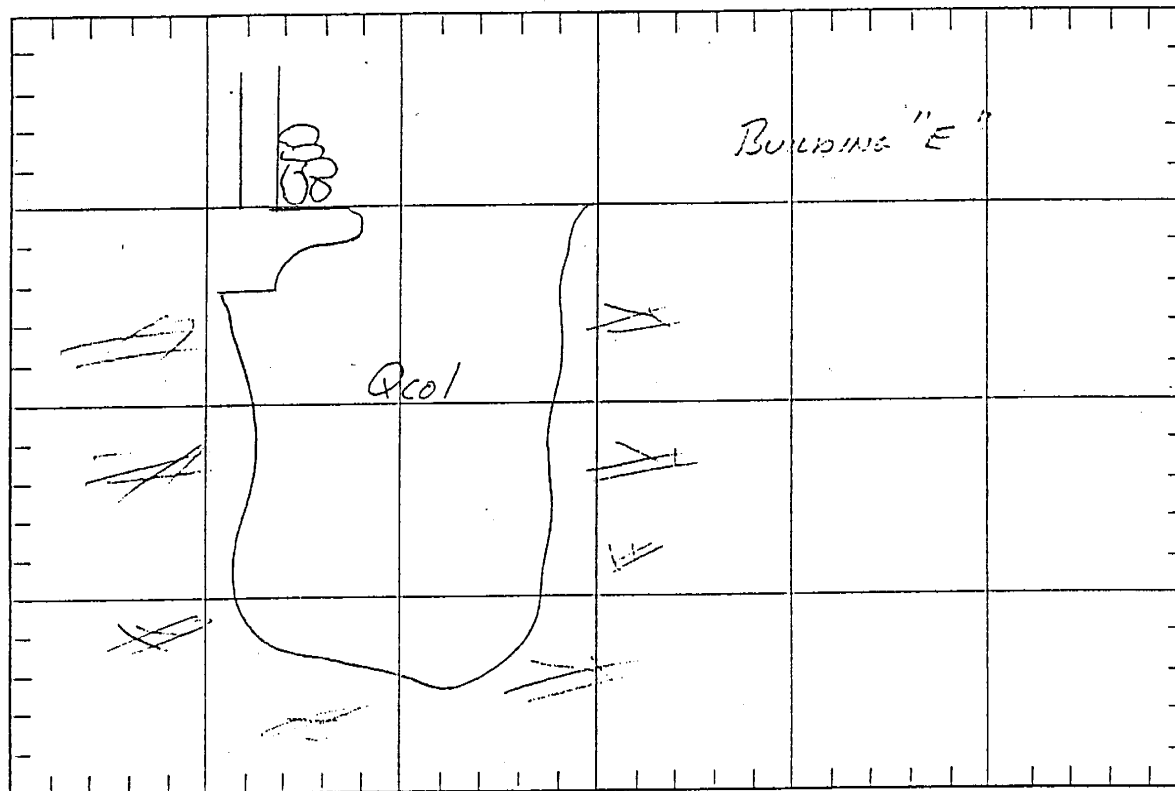
Trench Bearing _____



RECORD OF SUBSURFACE EXPLORATION

W.O. # 27190				Project: Rick Lawrence		Date: 12-13-91	TEST PIT # 9
USCS	Samples	Moisture %	Dry Density (pcf)	Depth (ft.)	Logged by: D.H.		Elevation:
					Equipment: Hand		Samples: B: Bag R: Ring T: Tube
					Description of Earth Materials		
CL	R	14	93	1	Dark Brown Sandy Clay, Very Moist, Soft (Qcol)		
				2			
ML	R	14	116	3	Red Brown Clayey Sandy Silty, Moist to Very Moist, Firm to Stiff (Qcol)		
				4			
				EOB @ 4.5' No Water, No Caving			

CROSS SECTION SKETCH



Scale 1" = 2'

Trench Bearing _____



RECORD OF SUBSURFACE EXPLORATION

W.O. # 27190

Project: Rick Lawrence

Date: 12-13-91

TEST PIT # 11

USCS

Samples

Moisture %

Dry Density
(pcf)

Depth (ft.)

Logged by: D.H.

Elevation:

Equipment: Hand

Samples: B: Bag R: Ring T: Tube

Description of Earth Materials

CL

R

10

102

1

Dark Brown Silty Sandy Clay, Moist, Moderately Firm to Firm,
Minor Gravels near Surface

(Qc01)

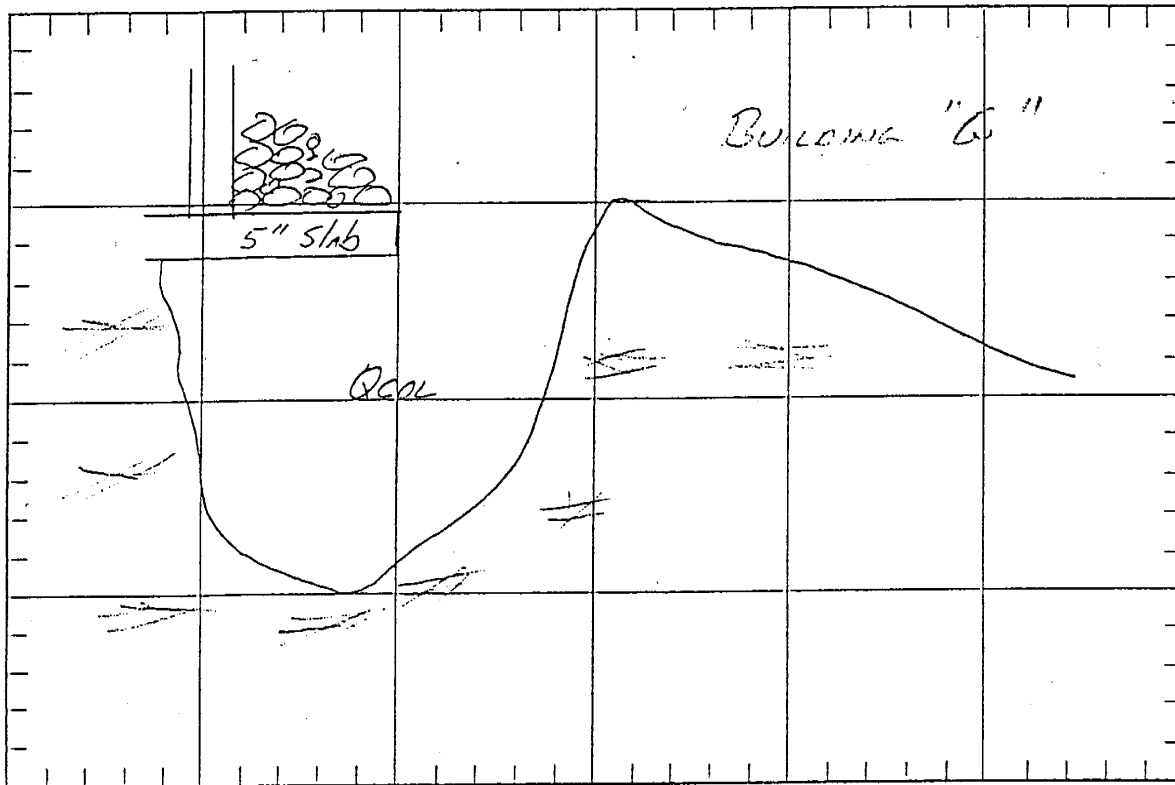
2

3

4

EOB @ 4'
No Water, No Caving

CROSS SECTION SKETCH



Scale 1" = 2'

Trench Bearing _____

STRATA-TECH GEOTECHNICAL CONSULTANTS

Report Date:

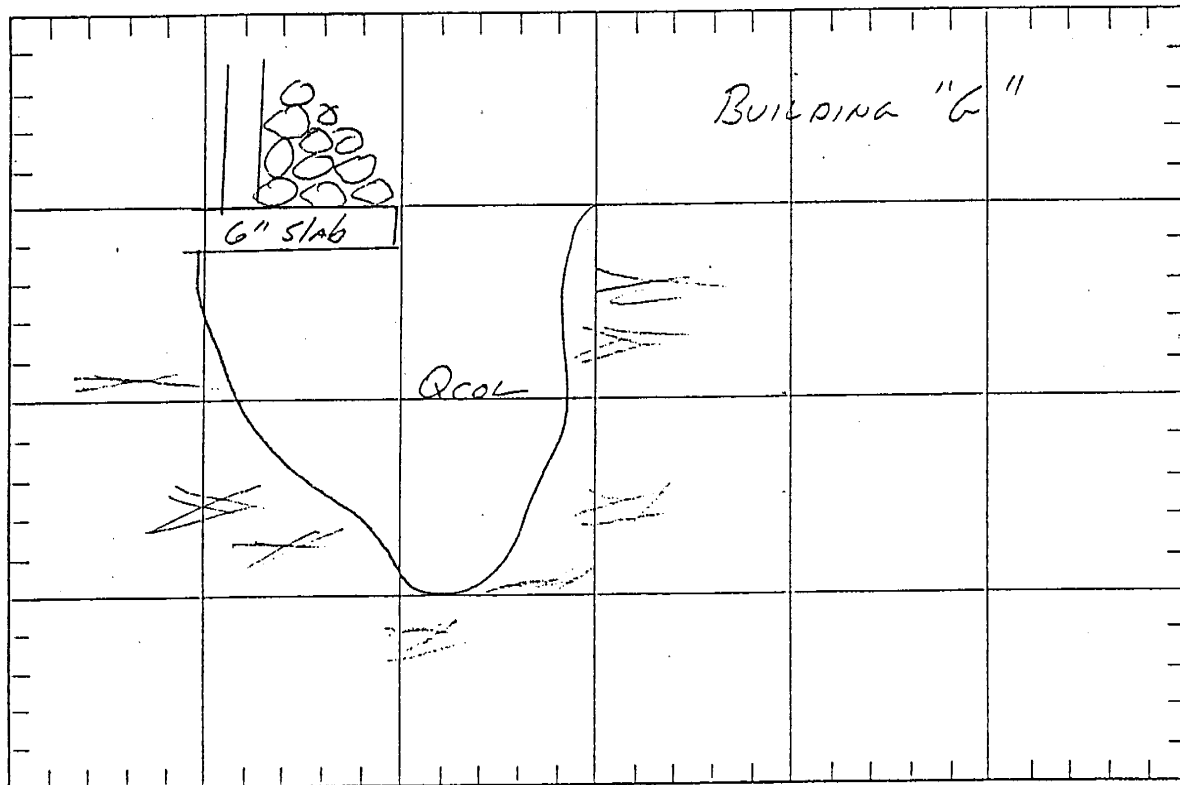
Plate: L



RECORD OF SUBSURFACE EXPLORATION

W.O. # 27190				Project: Rick Lawrence		Date: 12-13-91		TEST PIT # 12	
USCS	Samples	Moisture %	Dry Density (pcf)	Depth (ft.)	Logged by: D.H.		Elevation:		
					Equipment: Hand		Samples: B: Bag R: Ring T: Tube		
					Description of Earth Materials				
CL	R	11	110	1	Dark Brown Silty, Sandy Clay, Moist, Firm, Minor Gravels (Qc01) EOB @ 4' No Water, No Caving				
			2						
			3						
			4						

CROSS SECTION SKETCH



Scale 1" = 2'

Trench Bearing _____

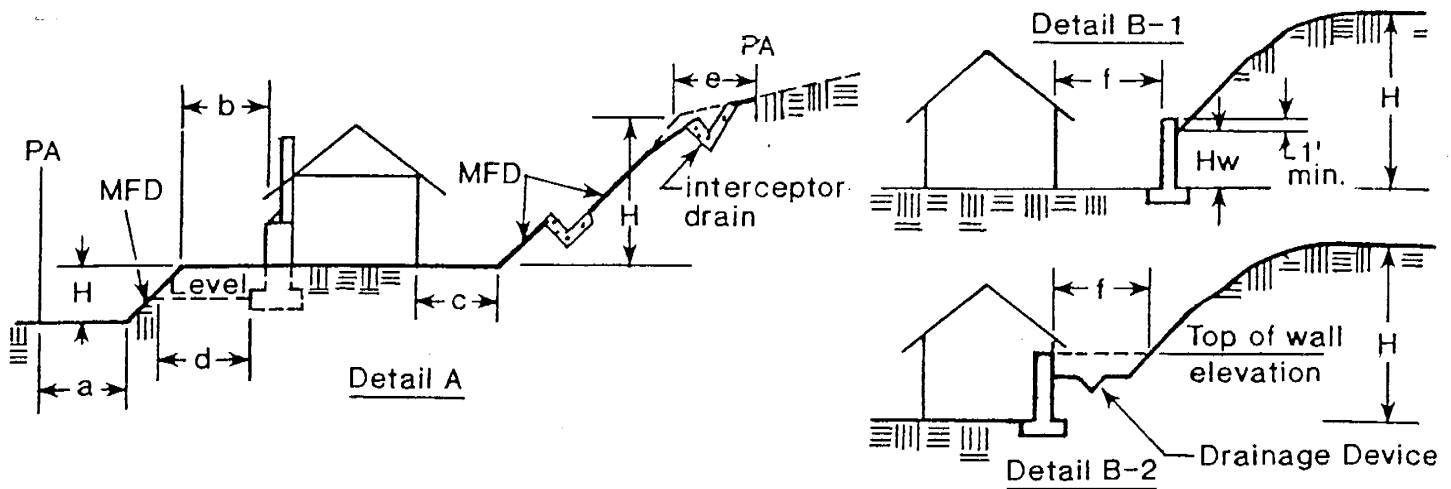
STRATA-TECH GEOTECHNICAL CONSULTANTS

Report Date:

Plate: M



Figure 2



Min. Setback from Adjacent Slope					
H(hgt.) Feet	a	b	c	d	e
0-6	3'	7'	3'	5'	1'
6-14	5'	7'	H/2	H/2 5'min.	H/5
14-30	5'	H/2 10'max.	H/2	H/2 10'max.	H/5
+30	5'	10'	15'	10'	6'

Table A

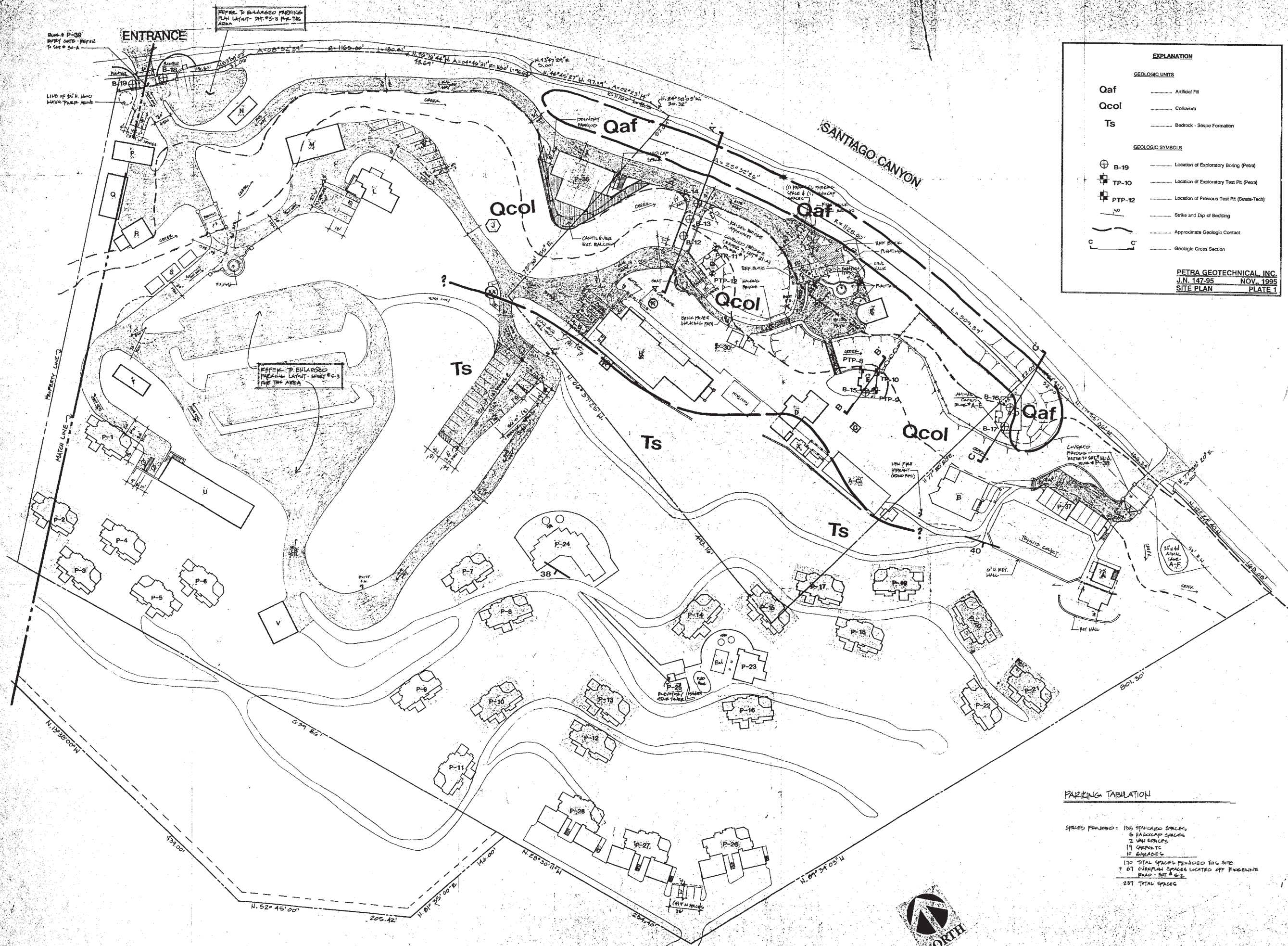
H(hgt.) Feet	Max. Hw	Min. Setback f
0-6	3'	3'min.
6-12	H/2	H/2
12-30	6'	H/2
+30	6'	15'

Table B

NOTES:

1. PA means permit area boundary and/or property line; MFD means manufactured surface.
2. Setbacks shall also comply with applicable zoning regulations.
3. Table A applies to manufactured slopes and 2:1 or steeper natural slopes. Setbacks from natural slopes flatter than 2:1 shall meet the approval of the Building Official.
4. "b" may be reduced to less than 5' minimum if an approved drainage device is used; roof gutters and downspouts may be required.
5. "b" may be reduced to less than 5' if no drainage is carried on this side of structure and if roof gutters are included.
6. If the slope between "a" and "b" levels is replaced by a retaining wall, "a" may be reduced to zero and "b" remains as shown in Table A. The height of the retaining wall shall be controlled by zoning regulations.
7. "b" is measured from the face of the structure to the top of the slope.
8. "d" is measured from the lower outside edge of the footing along a horizontal line to the face of the slope. Under special circumstances "d" may be reduced as recommended in the approved soil report and approved by the Building Official.
9. "f" may be reduced if the slope is composed of sound material that is not likely to produce detritus and is recommended by the soil engineer or engineering geologist and approved by the Building Official.
10. "a" and "e" shall be 2' when PA coincides with arterial or local street right of way and when improved sidewalk is adjacent to right of way.
11. "e" shall be increased as necessary for interceptor drains.





EXPLANATION

GEOLOGIC UNITS

Qaf Artificial Fill

Qcol Colluvium

Ts Bedrock - Sages Formation

GEOLOGIC SYMBOLS

⊕ B-19 Location of Exploratory Boring (Petr)

⊕ TP-10 Location of Exploratory Test Pit (Petr)

⊕ PTP-12 Location of Previous Test Pit (Strata-Tech)

40 Strike and Dip of Bedding

..... Approximate Geologic Contact

..... Geologic Cross Section

PETRA GEOTECHNICAL, INC.
J.N. 147-95 NOV. 1995
SITE PLAN PLATE 1



OWNER: RICK LAWRENCE
19191 LAWRENCE CANYON
STAR ROUTE BOX 1038
ORANGE, CA 92667

RANCHO LAS LOMAS
ORANGE, CALIFORNIA

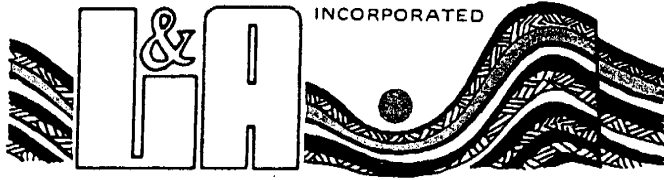
PARKING TABULATION

SPACES PROVIDED:	130 STANDARD SPACES
	6 HANDICAP SPACES
	2 VAN SPACES
	19 CARPARKS
	10 GARAGES
170 TOTAL SPACES PROVIDED THIS SITE	
7 67 OVERLAP SPACES LOCATED OFF PROPERTY	
237 TOTAL SPACES	

SITE PLAN



LEIGHTON and ASSOCIATES



SOIL ENGINEERING

TESTING

GEOLOGY

ENVIRONMENTAL SCIENCES

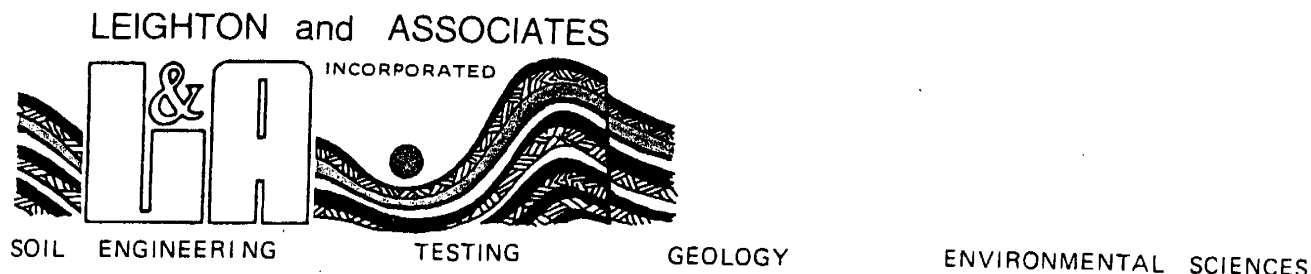
PRELIMINARY GEOLOGIC RECONNAISSANCE AND
ENVIRONMENTAL IMPACT ASSESSMENT OF
LAWRENCE PROPERTY,
SANTIAGO CANYON ROAD,
ORANGE COUNTY, CALIFORNIA

February 4, 1983

Project No. 1820248-01

Prepared for:

MR. R. L. LAWRENCE
Star Route, Box 1038
Orange, California 92667



February 4, 1983

Project No. 1820248-01

TO: Mr. R. L. Lawrence
Star Route, Box 1038
Orange, California 92667

SUBJECT: Preliminary Geologic Reconnaissance and Environmental Impact Assessment of Lawrence Property, Santiago Canyon Road, Orange County, California

In accordance with your request and authorization, we have completed a preliminary geotechnical assessment study of the subject property, as outlined in our proposal of April 23, 1982. Submitted for your review and use in the preparation of an environmental impact report (as well as for general land-use planning guidance) are six copies of our report which documents the research, analysis of field reconnaissance, and evaluation of the potential geologically related environmental impacts, hazards and constraints to the proposed land uses, or possible improvements related to them. This report, which was prepared in accordance with the CEQA and Orange County guidelines for EIRs, summarizes the findings of our analyses and presents possible mitigation measures to minimize the potentially adverse impacts identified.

We appreciate the opportunity to be of service to you. Should you have any questions regarding this report or require further information, please do not hesitate to contact the undersigned.

Respectfully submitted,

Richard Lung, Vice President
Principal Engineering Geologist, EG 111

Reviewed by: Bruce R. Clark
Principal Engineering Geologist, EG 1073

RL/BC/sdb

Attachments: Figures 1, 2 and 3
Tables 1 and 2
Appendix A

Distribution: (6) Addressee

1.0 SUMMARY OF FINDINGS AND CONCLUSIONS

- Our geotechnical analysis of the site and assessment of potential environmental impacts posed by the proposed future commercial, and existing agricultural land uses, have revealed no geologic, soil or hydrogeologic constraints, hazards or problems sufficiently serious to preclude the proposed uses or cause unavoidable adverse environmental impacts, provided that currently applicable codes are observed and appropriate construction methods are utilized during development or improvement of the site.
- Although there are no existing landslides or major slope stability problems anticipated to affect the property, surficial slope failures (e.g., mudflows or accelerated erosion) could affect the steeper portions of the site, particularly if runoff is not appropriately controlled. Site-specific geotechnical investigations will be necessary to evaluate and review proposed development plans, and to recommend mitigation measures, if necessary.
- Additional on-site sewage disposal systems required by future dwellings or other facilities are considered feasible on a limited or interim basis, but should be connected to a sanitary sewer when it becomes available.
- No special fault or seismic shaking hazards are anticipated to affect the site or require special building restrictions or building designs.

2.0 INTRODUCTION

2.1 Objective and Scope

The purpose of our study was to provide a preliminary geotechnical assessment of potential environmental impacts of the proposed commercial and residential/agricultural land uses of the subject property (known as Rancho Las Lomas), and to provide necessary documentation for a zone change application. This report identifies potential geologic, seismic, soil and hydrologic hazards and constraints; evaluates potential impacts on mineral resources; and presents possible mitigation measures, where necessary. The scope of our studies included the following:

1. Review of pertinent published maps and reports, including the Orange County Seismic Safety Element and the Foothill/Trabuco Plan EIR; refer to Appendix A for a complete list of references.
2. Analysis of sequential stereoscopic aerial photographs to document slope and site history.
3. Geological site reconnaissance and field mapping.
4. Data analysis, impact assessment and report preparation.

2.2 Proposed Land Use

It is our understanding that the existing agricultural zoning of the property is proposed to be changed, in part, to a commercial zoning to accommodate the private zoo usage contemplated. The commercial zoning would be applied to the smaller portion of the property (shown as the 4[±]-acre part, designated Parcel I on Figure 2), and the residual parcel would remain agriculturally zoned, with the possibility that residential structures might be proposed on it in the future.

2.3 Site Description

The subject property, comprised of approximately 21.4 acres, is located along the west side of Santiago Canyon Road (formerly Modjeska Road), about 1,000 feet north of its intersection with Live Oak Canyon Road (Cook's Corner); refer to the Index Map, Figure 1, which depicts the site location and the general topography of the area. Details of the site topography are illustrated on the base map of the Geologic Map (Figure 2).

The natural terrain of the site is characterized by gentle to moderately sloping hillsides adjoining the canyon bottom of Aliso Creek in the eastern one-third, and steeper, more rugged hillside ascending westward in the remaining two-thirds of the site. The maximum topographic relief (elevation difference between the highest and lowest points) within the property is approximately 285 feet. Slope gradients range from about 20 percent (5 horizontal to 1 vertical ratio) near the canyon bottom and on the gentle knoll in the north portion of the property, to about 50 percent (2 horizontal to 1 vertical ratio) and locally steeper on the ridge flanks.

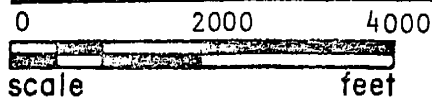
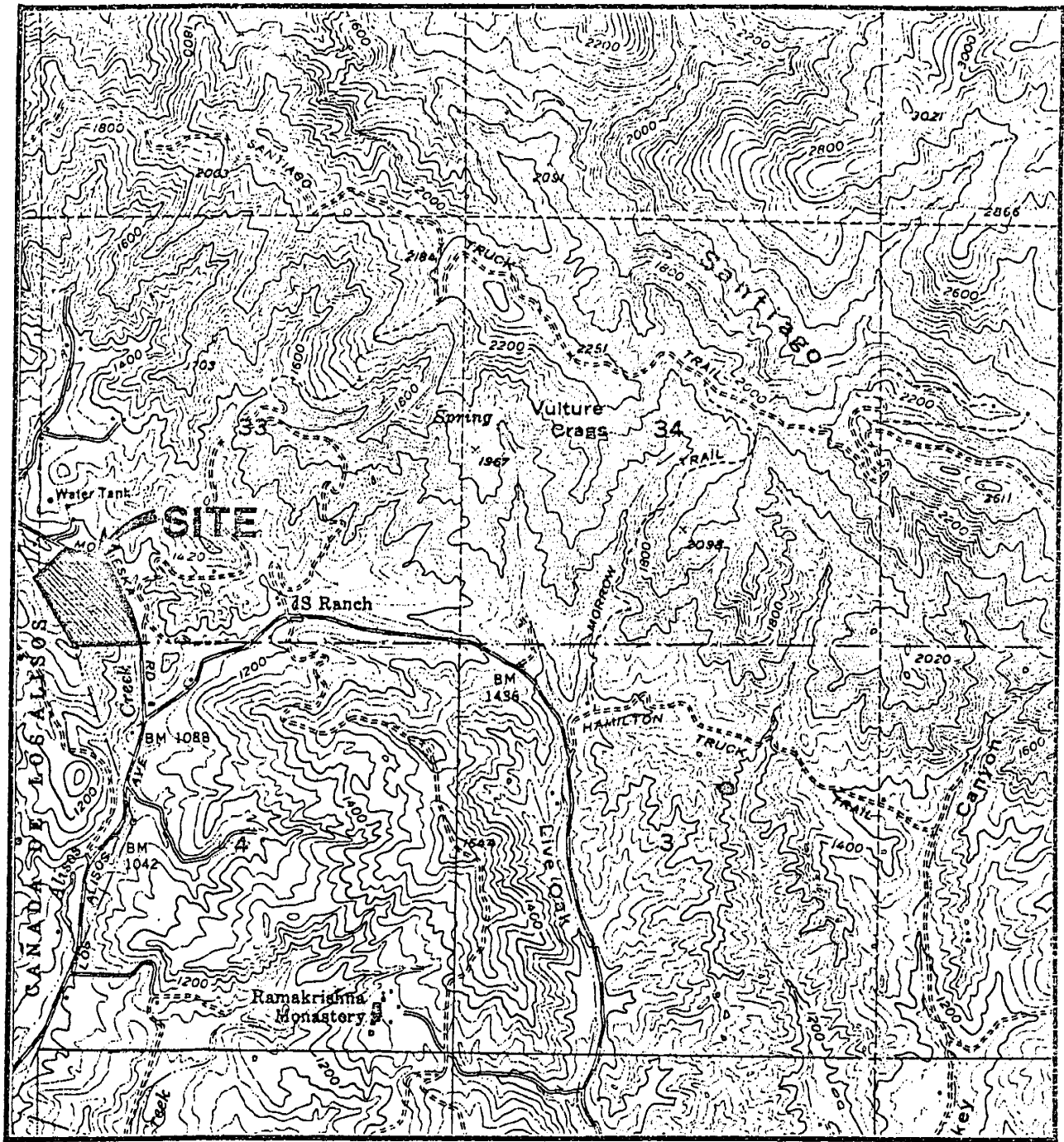


Figure 1

INDEX MAP
OF
LAWRENCE PROPERTY
SANTIAGO CANYON ROAD
ORANGE COUNTY, CALIFORNIA

BASE MAP: U.S.G.S. Santiago Peak Quadrangle

Native vegetation covering most slopes includes annual grasses, scrub oak and chaparral, with some large trees on the steeper flanks of the ridge in the central and south portions of the site, and along the canyon bottom. Water-seeking plants (phreatophytes) are common near the creek, where shallow groundwater tends to be present following seasonal rains. Citrus and various other types of fruit or nut trees are planted in the more gently sloping portions of the property.

Man-made features on the site include two dwellings which are presently occupied and a network of paved and dirt access roads in good repair throughout the site. Portions of the site are presently being used for agricultural purposes and are accompanied by irrigation lines, small drainage dikes and a small water storage tank. Fence lines skirt the perimeter of the property and portions of the area surrounding the existing dwellings.

2.4 Historic Activities

In the past, the subject site has been used for cattle grazing. Agricultural land use was probably initiated about the time the first dwelling was constructed, which is estimated to be in the 1950s. Comparison of old (1939) and new (1983) aerial photographs indicates that the Santiago Canyon Road has been realigned and apparently widened slightly, probably sometime after 1974. Recently, there has been a considerable amount of grading just southwest of the site (higher on the ridge), which is reportedly for a housing development.

3.0 GEOLOGIC SETTING

3.1 Regional Geology

The study area is situated on the southern flank of the Santa Ana Mountains, in the northwest Peninsular Range Province of southern California. This region is composed of a sequence of marine to nonmarine sedimentary strata, ranging in age from late Cretaceous to early Miocene, which were uplifted and tilted southwestward at moderate to steep angles.

Significant faults in the region include the Aliso fault (two miles east of the site), and a zone of unnamed faults along the projected trace of the Cristianitos fault, passing west of the site within 1.0 mile. More distant major potentially active and active faults include the Whittier-Elsinore, Newport-Inglewood, San Jacinto, Sierra Madre and San Andreas faults. No significant faults are known to transect the subject properties.

3.2 Bedrock Formations

Bedrock at the site consists of a transitional, interfingering portion of the Vaqueros and Sespe Formations designated "Tvs" on the accompanying Geologic Map. The contact mapped by others (see Reference 8, Appendix A) was not apparent by surface mapping of the site and exposures in the westerly portion of the site are characteristic of Sespe Formation in the area, rather than the Vaqueros Formation, as mapped by others. The bedrock is comprised of light-colored, massive sandstone with thick-bedded siltstone and local clay-rich interbeds. Although the two formations are of similar composition, the older Sespe Formation is of continental origin and lacks marine (mollusk) fossils, which are sometimes found in the Vaqueros Formation.

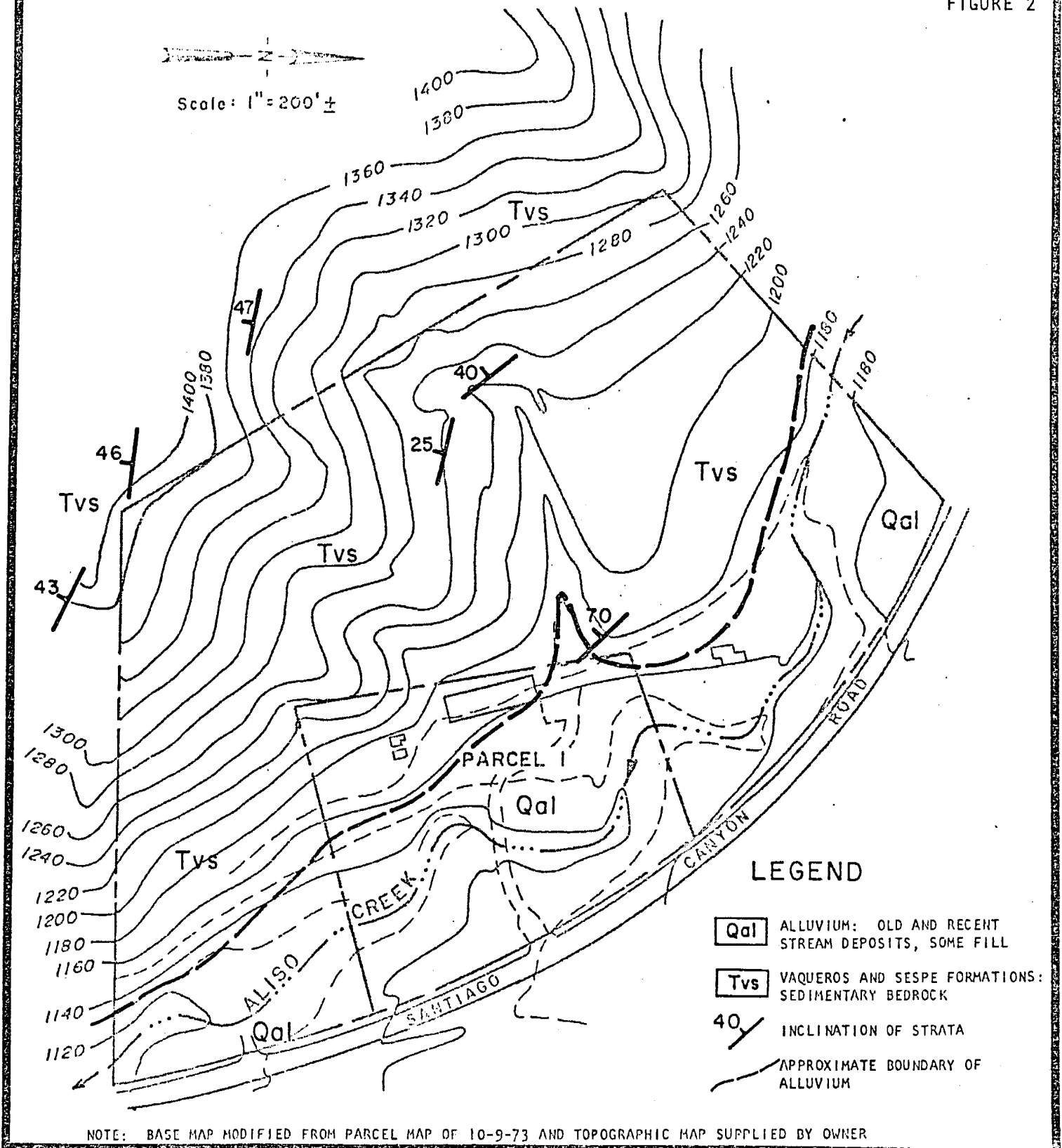
3.3 Soil Types

Alluvium and slope wash deposits (Qal on Figure 2) occur in the valley bottom areas in the central part of the site. Ephemeral tributaries contain little or no alluvium. These deposits are of varying composition, generally unconsolidated, and of variable thickness greater than 4 feet. Included in this mapped unit are minor amounts of younger alluvium in the active stream channels and local occurrences of older alluvium along the flanks of Aliso Creek and below Santiago Canyon Road. Some fill is present along access roads and is probably associated with the planted grove areas and the road realignment grading.

The residual surface soils or topsoils are generally less than 2 to 3 feet thick. In terms of agricultural classification, two main types of surface soils have been mapped by the Soil Conservation (Reference 22). The most predominant types at the site are the Alo clay, which mantles nearly all of the steeper slopes, and the Sorrento loam, developed on the alluvial and slope wash deposits lining the canyon bottom areas. The clay typically has a low permeability and a high shrink-swell potential. Such characteristics are not particularly desirable from an engineering standpoint and can present some building site development limitations (Reference 22). The loam generally has less development constraints and is more

GEOLOGIC MAP
LAWRENCE PROPERTY
SANTIAGO CANYON ROAD, ORANGE COUNTY

FIGURE 2



suited for agricultural purposes than the clay. Both, however, are rated as having a high erosion potential where the soil is bare.

3.4 Geologic Structure

The bedrock formations underlying the subject property represent a relatively simple geologic structure, forming a consistent homoclinal sequence of strata inclined southwestward. The bedding trends (strikes) generally northwest and inclines (dips) southwestward between about 25 and 45 degrees, but locally as steep as 70 degrees. Local steepening of beds probably results from broad warps in the otherwise homoclinal structure, as is characteristic of the general area. Measurements of the bedding inclination within and upslope of the property are shown on Figure 2.

3.5 Faults

No major or active faults are known or suspected to cross the subject property, based on our field reconnaissance and referenced maps or reports. The 4-S Ranch fault, a major fault once believed to exist just northeast of the site (References 9 and 16) has been reinterpreted as a normal formation boundary (R. Miller, California Division of Mines and Geology, personal communication).

Although fractured bedrock material is exposed in the upper 5 to 10 feet of canyon walls in the central portion of the site, it appears to result from weathering and probable slope creep, rather than from faulting.

The nearest major faults are approximately 800, 4,500 and 5,500 feet west of the site. The latter two are probable extensions of the Cristianitos fault zone and may be considered potentially active, based on data further south, along the zone (refer to Figure 3). In the absence of known active faults in the vicinity, however, there have been no Alquist-Priolo Special Studies Zones established on or near the site. Therefore, no mandatory requirement for the special investigation of fault rupture hazards imposed by such zones applies to the subject property.

3.6 Seismicity

Past earthquake activity, as documented by maps plotting earthquake epicenters, their associated magnitudes, and other seismic data, serves as a guide to probable future seismic activity at a given location, provided that the length of record is long enough to be representative. In some cases, particularly for assessing the seismic risk associated with potentially active faults (for which the geologically recent seismic record is not typical of past activity), another method of analysis is required.

The major active faults in southern California which are considered likely sources of future seismic activity that might produce significant ground shaking at the site are the Whittier-Elsinore, Newport-Inglewood, San Andreas, San Jacinto and Sierra-Madre. Refer to Figure 3 for the location of these faults and epicenters of significant earthquakes relative to the site. A review of earthquake epicenters by Morton and others (Reference 10) indicates a series of moderate earthquakes with magnitudes of 4.0 and 5.5 occurred just northeast of the site. This cluster of epicenters has been attributed to seismic activity on the Whittier-Elsinore fault in

1938 and most likely resulted in significant ground shaking at the site. The map also indicates a small 2.0 magnitude event just southwest of the site along the Cristianitos fault zone; however, this has not been generally accepted as proof that the fault is active.

Table I summarizes the seismic parameters of each of the major active faults listed above. In the event that the largest probable earthquake occurs on any of these faults, the seismic shaking to be felt at the site will depend upon the magnitude of the earthquake, the distance to the epicenter, and the site response characteristics. The key seismic parameters which influence the site response and the design of structures to resist ground motion are acceleration, predominate period, and duration of strong shaking.

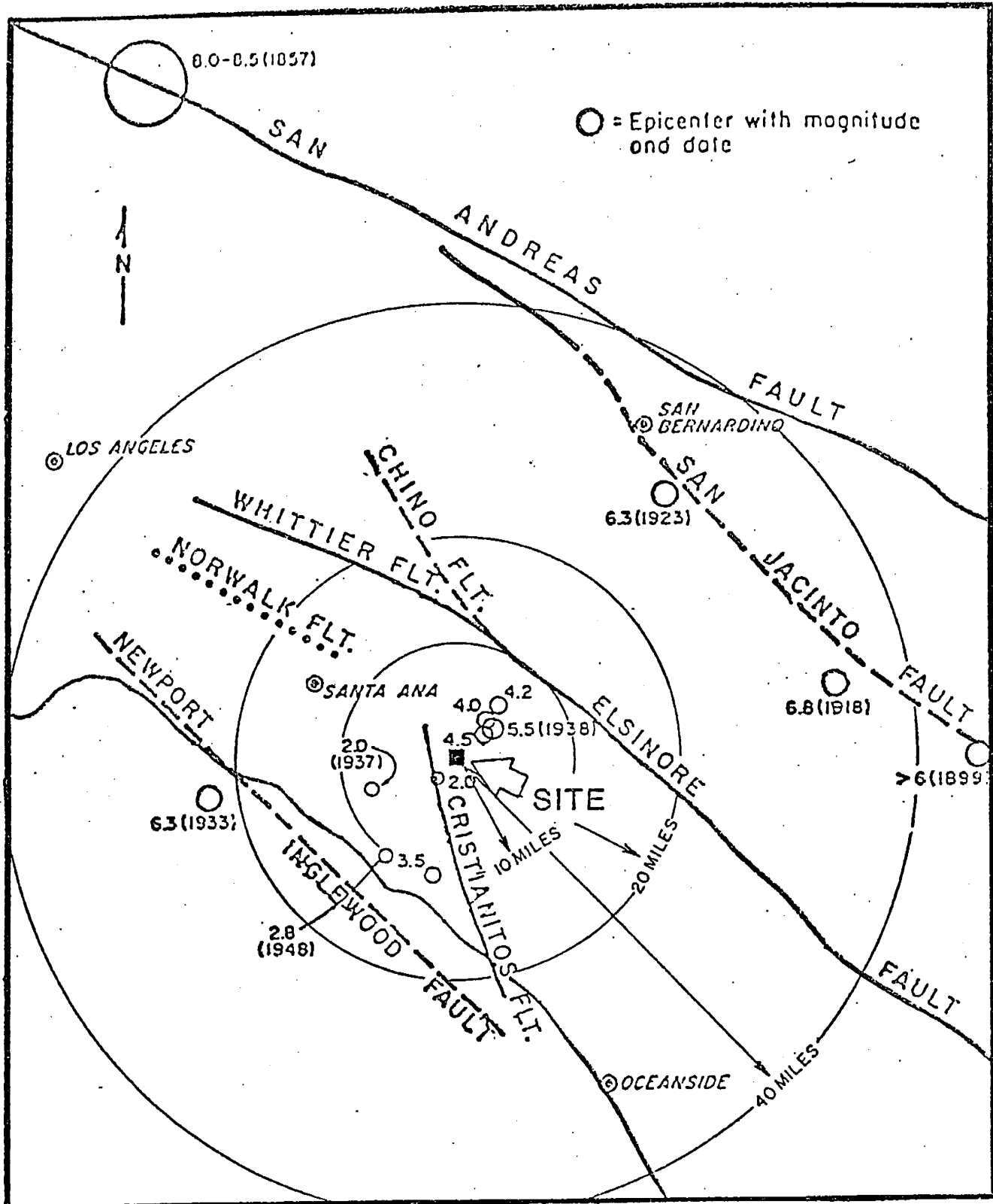
Additional information shown on Table I includes historic earthquakes and estimated recurrence intervals for maximum credible earthquakes (larger than maximum probable earthquakes which generally apply only when considering the design of critical structures, such as dams, nuclear reactors, or hospitals). Private residences, commercial buildings, schools and nearly all other types of high-occupancy structures are designed for the maximum probable event, or in accordance with current building code requirements.

3.7 Slope Stability

Existing slope stability conditions within the site are generally favorable, as evidenced by the absence of significant landslides. The on-site bedrock formation, however, is susceptible to erosion and relatively shallow, slump-type failures which typically affect the near-surface soils or weathered bedrock materials, particularly where they underlie the steeper slopes in the westerly portion of the site (refer to Figure 2). Erosion can occur along existing drainage channels and unpaved access roads, particularly from heavy runoff during the rainy season.

An earlier published map (Reference 9) indicated the possible existence of a large landslide underlying the more gently sloping knoll in the north portion of the site. Subsequent mapping by the state (Reference 8) and by us during our recent reconnaissance revealed no supportive evidence that a slide is present.

The composition and geologic structure of the underlying bedrock formations are generally not conducive to major landslides, provided that zones of weakness (stratified clay and siltstone) and potentially adverse slope angles or directions are recognized and mitigated in the development grading. Although the majority of the bedrock is not considered to be unusually prone to sliding, clay-rich beds within the bedrock may be susceptible to sliding, particularly if erosion or excavations have undercut the strata, leaving the slope without sufficient lateral support.



MAP OF FAULTS AND REGIONAL SEISMICITY
OF

LAWRENCE PROPERTY
(FROM MORTON AND OTHERS, 1974)

TABLE 1
SEISMIC PARAMETERS FOR
LAWRENCE PROPERTY
ORANGE COUNTY, CALIFORNIA

POTENTIAL CAUSATIVE EARTHQUAKE FAULT	CLOSEST DISTANCE FROM FAULT TO SITE	LENGTH OF FAULT	RICHTER MAGNITUDE OF HISTORICAL EARTHQUAKE	APPROXIMATE AGE OF MOST RECENT SURFACE DISPLACEMENT	MAXIMUM CREDIBLE EVENT		MAXIMUM PROBABLE EARTHQUAKE (DESIGN EARTHQUAKE)			
					RICHTER MAGNITUDE	MEAN ESTIMATED RECURRENT INTERVAL IN YEARS	RICHTER MAGNITUDE	PEAK HORIZONTAL GROUND ACCELERATION AT SITE (FRACTION OF GRAVITY) (NOTE 4)	PREDOMINATE PERIOD AT SITE (IN SECONDS) (NOTE 5)	DURATION OF STRONG SHAKING AT SITE (IN SECONDS) (NOTE 6)
SAN ANDREAS FAULT (SOUTH OF GARLOCK FAULT)	61 km 38 mi	500 km 310 mi	8.25+ (1857) 6.5 (1948)	HISTORIC (1857 and 1948)	8.5	M8=125-225 (NOTE 7)	8.3	.21	0.48	23
SAN JACINTO FAULT	50 km 31 mi	440 km 274 mi	7.0 (1899) 6.5 (1968)	HISTORIC (1899 and 1968)	7.5	M6 = 4-10 M7 = 40-100 M8 = 400-1000	7.2	.15	0.35	24
WHITTIER-ELSINORE-AGUA CALIENTE FAULT	11 km 7 mi	260 km 162 mi	5.5 (1938) 6.0± (1910)	HISTORIC (1910)	7.5	M6 = 20-90 M7 = 200-900 M8 = 3000-9000	6.7	.41 (.27)*	0.30	22
SIERRA MADRE FAULT	47 km 29 mi	90 km 56 mi	6.4 (1971)	HISTORIC (1971)	7.0	M7 = 5000 (NOTE 3)	6.5 (NOTE 3)	.10	0.30	14
NEWPORT-INGLEWOOD FAULT	27 km 17 mi	80(+) km 50(+) mi	6.3 (1933)	HISTORIC (1933, unconfirmed)	7.0	M7 = >300(?)	6.5	.20 (.13)*	0.22	15

- Note 1. Postulated maximum rupture length based on L/2 (maximum credible earthquake), and L/5 (maximum probable earthquake).
- Note 2. After D. Lamar, 1973.
- Note 3. After R. Crook, B. Kamb, C. Allen, M. Payne and R. Proctor, 1978.
- Note 4. After Schnabel and Seed, 1973. The repeatable high ground acceleration (*), taken as 65 percent of the peak acceleration, for sites within 20+ miles of the epicenter (after Ploessel and Slosson, 1974), may be more applicable for design analysis.
- Note 5. After Seed, Idriss and Kiefer (1969).
- Note 6. After Bolt (1973).
- Note 7. After Sieh (1981), for "south central section" of fault near Cajon Pass.

4.0 RESOURCES

4.1 Groundwater and Surface Runoff

The site is in the upper part of the Aliso Creek watershed, which is approximately 15 miles from the ocean where the creek empties. Considering the relatively rapid runoff and probably minimal percolation which is typical of this terrain, groundwater resources within the property are most likely limited to perched water zones within the alluvium and saturated bedrock zones immediately beneath the alluvium. Although no runoff was observed in the creek at the time of our reconnaissance, it and other drainage courses carry seasonal flows. An old water well with a windmill next to the northerly dwelling reportedly provides small amounts of water for irrigation purposes. We understand that there is a spring on the hillside above the property.

The principal groundwater resources within the Aliso Creek watershed are located about 3 miles south, along El Toro Road, where the alluvium is more extensive. There, water well records indicate that water levels range from about 5 to 25 feet from the surface in past years, with a few being dry in the summer months (Reference 2).

4.2 Mineral Deposits

According to the California Division of Mines and Geology (Reference 9) the nearest sites of economically significant mineral resources are located across Santiago Canyon Road, about ½ mile north of the subject property. There, two mines (the Serrano Mine, and later the Schoeppe Mine) were in operation intermittently from 1926 to 1975, when all activity was ceased. Both produced clay and silica sand from the upper part of the Silverado Formation. This formation does not underlie the subject property, and there is no known mineral resources of economic importance contained in the formation which is present.

5.0 PRINCIPAL GEOTECHNICAL HAZARDS, CONSTRAINTS, IMPACTS AND POSSIBLE MITIGATION MEASURES

5.1 Factors or Geologic Problems Evaluated

This section presents, in summary form, the principal geotechnical factors that were considered and rated on a subjective scale, comparing the subject site with the range of hazard severity which is generally representative in southern California, refer to Table 2, which presents, in matrix form, the hazard rating and possible mitigative measures). A discussion of the principal geologic and hydrogeologic constraints follows.

5.2 Fault Displacement

In the absence of any major faults crossing the site, the hazard of ground rupture from fault displacement is considered to be nil. Therefore, no special fault studies or construction limitations due to fault movement potential apply to the subject site. Subsequent geologic site investigations and inspections required by the County at various development stages should provide appropriate mitigation measures if a major fault is found within the site.

5.3 Ground Shaking

Although moderate intensities of seismic ground shaking can be anticipated at the site (either from smaller earthquakes located relatively close, or from larger earthquakes farther from the site), the effects are expected to be satisfactorily mitigated by conformance with the latest (1982) Uniform Building Code, the Orange County Building Code, or recommendations of the Structural Engineers Association of California for seismically resistive design of structures.

5.4 Liquefaction and Related Ground Failure Phenomena

Secondary earthquake hazards, such as liquefaction, flow landsliding, seismically induced settlement, and ground lurching or cracking (shown as "ground rupture" in Table 2) are generally associated with relatively high intensities of ground shaking, shallow groundwater conditions and the presence of loose sandy soils or alluvial deposits. Because of probable soil conditions which underlie most of the site, and taking into account the moderate ground shaking intensities which could occur, such ground failure hazards are rated as slight, even though shallow groundwater could be present locally (along the canyon bottom) during and shortly after the rainy seasons. Therefore, no special mitigation measures, other than construction in accordance with code requirements is expected to be necessary. The detailed soil investigation required at the parcel map or building plan stage should provide sufficient data to permit a more definitive evaluation of the hazard.

LAWRENCE PROPERTY

TABLE 2. CHECKLIST OF GEOTECHNICAL HAZARDS AND POTENTIAL MITIGATION MEASURES
(MODIFIED FROM CDMG NOTE 46)

G E O L O G I C P R O B L E M S		. D E G R E E O F H A Z A R D O R P R O B L E M				P O S S I B L E M I T I G A T I O N M E A S U R E S		
PROBLEM	ACTIVITY CAUSING PROBLEM	NO/E	SLIGHT	MODERATE	SEVERE	CODE CONFORMANCE	CODE CONFORMANCE + SPECIAL WORK*	ADVANCE PLANNING, AVOIDANCE, RESTRICTIONS
EARTHQUAKE DAMAGE	FAULT MOVEMENT	X	X			X		
	LIQUEFACTION	X	X			X		
	LANDSLIDES		X			X		
	DIFFERENTIAL COMPACTION/ SEISMIC SETTLEMENT	X	X			X		
	GROUND RUPTURE		X				X	
	GROUND SHAKING		X	X		X		
	TSUNAMI	X				N.A.		
	SEICHES	X				N.A.		
	FLOODING (DAM OR LEVEE FAILURE)	X	X			N.A.		
LOSS OF MINERAL RESOURCES	LOSS OF ACCESS		X			X		
	DEPOSITS COVERED BY CHANGED LAND USE		X			X		
	ZONING RESTRICTIONS		X			X		
WASTE DISPOSAL PROBLEMS	CHANGE IN GROUNDWATER LEVEL		X					X
	DISPOSAL OF EXCAVATED MATERIAL		X			X		
	PERCOLATION OF WASTE MATERIAL		X	X			X or	X
SLOPE AND/OR FOUNDATION INSTABILITY	LANDSLIDES AND MUDFLOWS		X	X			X	
	UNSTABLE CUT AND FILL SLOPES		X				X	
	COLLAPSIBLE AND EXPANSIVE SOIL		X	X		X		
	TRENCH-WALL STABILITY		X			X		
EROSION, SEDIMENTATION, FLOODING	EROSION OF GRADED AREAS			X		X		
	ALTERATION OF RUNOFF		X				X	
	UNPROTECTED DRAINAGE WAYS		X	X			X	
	INCREASED IMPERVIOUS SURFACES		X			X		
LAND SUBSIDENCE	EXTRACTION OF GROUNDWATER, GAS, OIL, GEOTHERMAL ENERGY	X	X			X		
	HYDROCOMPACTION, PEAT OXIDATION	X				X		
VOLCANIC HAZARDS	LAVA FLOW	X				N.A.		
	ASH FALL	X				N.A.		

*"SPECIAL WORK" CAN INCLUDE ADDITIONAL INVESTIGATION, SPECIAL SITE PREPARATION, OR SPECIAL FOUNDATIONS.

5.5 Tsunami, Seiche, and Inundation Hazard

No problems connected with tsunamis (seismically generated sea waves), seiches (seismically generated waves in lakes or reservoirs), or inundation caused by dam failure are expected to affect the site. The possible future pond in the tributary canyon northwest of the main house would not appear to pose a significant hazard if appropriately designed and constructed.

5.6 Slope Instability

The formations underlying the site have been grossly stable, as judged by the absence of landslides. But because of the moderate to steep topography which characterizes the west portion of the site, and the presence of surficial soil and weathered bedrock which may, in part, be susceptible to shallow failures (such as mudflows), careful analysis of future development plans will be necessary to appropriately mitigate potential slope or building site stability problems. This could require corrective measures if unstable materials would be exposed by grading, or underlie the existing slopes adjoining proposed improvements. Possible alternatives to such standard mitigation measures as buttress fills or retaining walls would be to modify proposed slopes to have shallower gradients, lower heights, or by reorientation of slope direction. Structures also could be relocated away from hazardous slopes, particularly if they cannot be stabilized feasibly.

5.7 Flooding, Erosion and Sedimentation

The hazard of flooding, along with erosion and sedimentation, resulting from storm runoff is rated as slight for most of the property, although these could range to moderate or severe along the main drainage courses during heavy runoff periods. According to the National Flood Insurance Program maps (Reference 12), the potential inundation area anticipated from a 100-year flood in Aliso Creek begins at Cook's Corner and extends southward, away from the subject site. The existing Parcel Map for the property (Reference 4), however, indicates a flood hazard and inundation zone along Aliso Creek, presumably representing a more frequent flood recurrence interval delineated in accordance with Orange County criteria. Appropriate mitigation for such flood hazards would be to locate all habitable structures outside of any delineated zone or other drainage course.

On graded slopes, the erosion and sedimentation potential should be slight if they are constructed and landscaped in accordance with code requirements. Adequate surface drainage and control devices will be necessary to appropriately mitigate the potential runoff, erosion and sedimentation problems adjoining and within the sites proposed for development.

5.8 On-Site Sewage Disposal

The principal constraints for the use of on-site sewage disposal systems will be the highly variable soil or bedrock formation permeabilities likely to be encountered within the site, and the topographic limitations of the steeper terrain. These constraints, and potential negative impacts (such as reduction of slope stability from subsurface infiltration of sewage effluent) can be overcome or mitigated by adequate investigation of the site and appropriate design of a system on a site-by-

site basis. We understand that the existing onsite sewage disposal systems (septic tank and leach lines) have performed satisfactorily. Future building sites on the hillside, where the soils are likely to be less permeable, may require larger or a different type of sewage disposal system.

Considering the relatively low housing density likely to be proposed, the groundwater degradation impact of private sewage disposal systems is expected to be minimal as regards the main groundwater resources located several miles downstream. However, their impact on the local groundwater in the canyon bottom could be more significant.

5.9 Other Hazards, Constraints, or Impacts

All other potential adverse factors affecting the site (listed on Table 2) either do not apply or are expected to be of no more than average severity and mitigable by use of designs, standards or procedures prescribed by applicable codes.

APPENDIX A

APPENDIX A

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AERIAL PHOTOS REVIEWED

<u>Year</u>	<u>Flight</u>	<u>Photo No.</u>	<u>Source</u>
1939	5925	69, 74	Fairchild
1983	82117	3, 4	Don Read